June 9, 1932

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Relief and Rebuilding

Two important developments of the last week, agreement on a tax bill and action to rehabilitate security values, prepare for the chief step in rebuilding business, which is the creation of employment. But the prospect of prompt action to this effect has been dimmed by preoccupation with charity measures. Legislative as well as business thinking is concerning itself with "direct relief." How this new trend affects the efforts for resumption of local public construction is discussed in an editorial in this issue.

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Also in this issue:

Parallel Trusses for Dome of Cincinnati Station

Pure Water from a Polluted Chinese River

Unemployment Relief and Civic Improvements

Testing Highway Guard Rails in Service

Suspension Bridge Stresses by Model Studies

Deep Shaft Elevators in Carlsbad Cavern

Testing Cast Iron Pipe for Columns and Cores

Sewage Disposal Trends in New York Metropolitan Area

Time to Resurface with **BRICK**

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F. E. Schmitt, Editor

Volume 108 — New York, June 9, 1932 — Number 23

Relief and Rebuilding

IMPORTANT constructive events marked the past week. Congress agreed on an effective revenue-producing tax bill, and good progress was made in organizing financial groups to take the lead in rehabilitating security values. These two achievements remove many fears and uncertainties, and, though their ultimate effect in restoring economic health must wait the test of time, they are good preparation for the next step, the creation of employment.

At the moment, however, progress in this direction is checked by preoccupation with relief plans. It is proposed to extend national charity aid either directly or through states to those who are suffering distress from the loss of their earning power. The pressure of necessity has forced the subject into the foreground, and destitution has become a national issue. In this sudden prominence of relief the major issue of bringing about business recovery through employment of idle workers on local public construction is being overshadowed.

ALL will agree that in so far as relief is an imperative necessity it should have prompt, well-considered and vigorous action. But there ought to be no confusion of thought about its effect. We need to be particularly clear about its sufficiency, its effect upon business recovery and its relation to the public-works employment movement, for many think of relief as all-sufficient.

It has been said that public-works construction is unnecessary if charity relief is provided. It has also been argued that the two are incompatible because the funds required for relief would make it impossible to provide money in support of construction. This reasoning merits examination.

Relief cannot render employment unnecessary, since it is only a palliative but never a cure. Relief measures would not restore work or earning power, being specifically intended to maintain the idle. If anything, they would bring about further decline in industry and commerce, since the wants that form the life of trade are lessened when large groups of people are reduced to a bare subsistence level under state charity.

THE inadequacy of relief becomes most evident when two of its effects are considered: first, that charity inevitably pauperizes; second, that it can contribute nothing to business revival. The pauperizing effect is probably least serious where relief is allocated as an unemployment pension. It is most harmful in the form of bread-line charity to the destitute, the more probable form. We can hardly face with equanimity the intensive destruction of the spirit of self-help and initiative that is bound to result from such an aid system. If only for this reason, relief is at best a short-time expedient, to be terminated as quickly as possibly by bringing into action measures that will *prevent* destitution. But a further reason is found in the fact that relief obviously cannot be maintained in the long run unless the economic organization is speedily restored to normal.

AS TO THE influence of relief upon business, it need only be pointed out that its effects are all negative. Not a single item of consumption is added, no new wants are created, and no fresh enterprise or initiative is introduced into the business organism.

It is hoped and believed by many that relief measures will render unnecessary any attempts to bring the country back to normal conditions, because business will revive spontaneously in time, if left to itself. Given a sufficiently long time, this is doubtless true, but we think the cost would be far too great. Just such temporizing and waiting during the year past is responsible for the depth of distress today, and continued temporizing can bring nothing but continued distress.

Last year the government declared that we must depend on spontaneous recovery and upon charity, and it uncompromisingly opposed the prosecution of public works to employ the idle. The results of this policy are with us today. The construction that was then being urged might now be spreading its benefits countrywide by taking off the peak of unemployment. Quite as much hinges on action or inaction now. If something effective is done to put local public construction under way once more, it will mean a reduced relief burden next winter and in all likelihood steady progress upward; if nothing is done, we continue to drift into an uncertain future.

THUS, contributing nothing to employment or the upbuilding of business, restoring no buying power but, on the contrary, depressing it by holding large numbers of people at lowest subsistence level, relief becomes anything but an aid to reconstruction. And finally, the argument of financial limitation has little weight in the light of the latest facts. Measures now before Congress, and the President's proposals as well, contemplate provisions to finance not only relief but also employment-producing work. Relief, it seems, will not absorb such large resources as to prevent action of constructive kind.

Whether or not we have national relief, then, every reason urges the continuance of efforts to restore normal conditions of life and business, for more than relief is required if the helpful possibilities of what was accomplished last week are to come to fruition. Until another way is found to recreate work and earning power, employment in public construction warrants full faith and support, and adequate financing of such construction therefore remains the leading issue.

NOTES OF THE WEEK

A Dole Well Invested

WISE expenditure of personal contributions for unemployment relief in Boulder, Colo., has converted a potential dole into a community investment. The task performed by this little city, as described in the present issue, is equally ready to hand and equally simple in performance in every town, large or small, in the nation. It requires only intelligent citizenship direction and cooperation with the public-works officials to capitalize in the form of community improvements a large part of every dollar contributed to the aid of those out of work and to give to those aided the self-respect coming from a dollar earned instead of received in charity. By not adopting the lazy way of the dole Boulder has created a better city and better citizens.

Continuing Good Work

THE CREDITABLE records made by state highway departments in providing winter road work to reduce unemployment are being continued into the spring and summer, according to employment figures collected by the Association of State Highway Officials. During April 39 states placed 7,180 miles of state road under contract for a total of $95,828,518, and 185,795 men had been given employment. This year for the same month in the same number of states the figures were respectively 3,874 miles, $38,524,278, and 205,221 men given work. The additional men have been given work by shortening the working day and staggering employment, commonly on the basis of three days' work a week. In general, wage rates have not been reduced. The extension of highway construction and maintenance as a possibility for increasing the distribution of work requires no better proof than these figures. In the face of them, administration statements at Washington that public-works construction is a neglible means of reducing unemployment merit little respect when applied to roadbuilding.

State Aid for Cities

A SOUND MOVE has recently been made by the state of Michigan in connection with the financial plight of many of its local governments. Because of a number of impending defaults, a state commission has been formed to act as a refunding advisory body for municipalities that have bonds maturing within the next five years. It is not to be presumed at the outset that state credit is to be made available for city refinancing, for the rules under which the new commission is to operate have not yet been formulated, but without question refunding in many cases will be greatly facilitated by the more capable fiscal administration to be made available. The significance of the move lies in the recognition by the state of an obligation to assist its creatures when they are in financial difficulties. The obligation exists not only as a moral responsibility but also as a matter of business prudence, for in critical times the state's own credit is involved in that of its cities, however independent it might be during normal periods. If state assistance must result in some degree of state supervision of local administrative policies, this outcome is infinitely to be preferred to the arbitrary dictation of private interests, which has too often in recent months been set as the price of continued financial support.

Reasoning by Hand

REPLACEMENT or supplementing of mathematical analysis by the hand-measurement process of model testing has to its credit many recent achievements. At times one is even tempted to believe that we are back to the methods of the civil engineering pioneers, who tried out their structures in model before building. But in fact, modern model work is essentially different from that of the early days. The "deformeter" analysis, fathered by Prof. George E. Beggs, illustrates the point, since it is in essence the solution of certain fundamental equations of mechanics that apply to the structure being studied. Something parallel to this was done in the suspension-bridge analysis that he and his co-workers applied to the Mount Hope bridge and, in slightly different form, to certain tentative Bay Bridge designs. Most illuminating is the finding that in certain respects the model can go beyond the reasonable range of algebraic analysis. Proper utilization of such possibilities should lead on to expansion of structural powers.

Notable Weight Reduction

MOVING dead weight has long been recognized as one of the major burdens upon mass transportation of passengers, especially upon the steam railroads. The dead weight carried per passenger has increased progressively with the development of the railroad car from the stagecoach prototype. For this reason the self-contained rail cars now being built in this country with frames made up almost entirely of structural shapes of thin-section stainless steel are notable. Weight-saving changes have been made in other parts of the body, the weight of the trucks has been reduced, and by the use of six wheels per truck the introduction of pneumatic tires has been made possible. The result is a remarkably easy riding car and one in which great saving in dead weight has been accomplished. This type of car should show the way toward like economies in buses and other railway cars.

Making Guard-Rail Progress

CHANGED conceptions of the purpose of the highway guard-rail call for new design thinking. In this new thinking the mind must go far beyond looking on the guard-rail as a mere marker to define the road limits and to warn the driver of danger, and also beyond any notion that its function is to act as a rampart which a colliding car cannot breach. These purposes remain, but they are subordinate to the main purpose, which is to turn the colliding vehicle safely back onto the road. To serve this purpose, the barrier must spring under the impact of the colliding car and must present a smooth gliding surface where it contacts with the car. Primarily, also, this surface should contact with the vehicle at those parts which carry the reacting thrust most directly to the vehicle framework. In emphasizing these objectives the tests described in this issue and the shrewd reasoning with which they are interpreted are an important contribution to highway design.

ENGINEERING NEWS-RECORD—JUNE 9, 1932

Dome Framing of Cincinnati Station. 817
Sewage Disposal Trends in New York. 820
Elevators in Carlsbad Cavern. 821
Investing Relief Funds 823
Testing Highway Guard-Rails. 824
Cast-Iron Pipe for Columns 827
Suspension-Bridge Model Studies. 828
Pure Water for Shanghai Settlements 833
Letters to the Editor 837
News of the Week 838

Parallel Trusses Carry Dome of Cincinnati Station

By O. S. Payzant
Engineering Associate, Fellheimer & Wagner, Architects and Engineers, New York City

A SEMI-DOME type of roof adopted for the station building of the union passenger terminal at Cincinnati, Ohio, presented special problems in the design and erection of the structural-steel framing. In this building the main floor is some 35 ft. above the normal ground elevation of the surroundings, thus giving the building a conspicuous and significant position fronting on a plaza at the head of a landscaped approach incline, as described in *Engineering News-Record* of Feb. 26, 1931, p. 348. In this plaza entrance the central part is reserved for the use of persons on foot and in automobiles. Triple loops at the sides of the plaza and passing through the station at lower elevations provide for street cars, buses and taxicabs. A driveway loop at the ground level serves the heavy trucking.

As the requirements for occupancy did not warrant a high or monumental building, it became necessary to adopt some lofty type of roof to give architectural prominence to the city front of the building. In the general layout the main floor or concourse is level with the train concourse that spans the tracks, and a semicircular plan for this floor was selected as permitting the arrangement of the various station utilities along the curved sides converging to the entrance of the train concourse. On account of this semicircular plan a truss-supported semi-dome roof was conceived as supplying adequate spaciousness for the interior and an imposing cylindrical or curved facade.

A SEMI-DOME steel-frame roof is an unusual structural feature of the new union station at Cincinnati. Parallel arched trusses, largely of H-section members and of varying spans, are placed at different elevations to fit the curved outline of the dome, while the highest and largest trusses carry a barrel-arch or cylindrical extension of the dome. This great station, built on filled ground in what was a waste-land section of the city, will serve seven trunk-line railroads with its twenty-two tracks (fifteen at present). Besides the $41,000,000 for the station and terminal, the several railroads will spend an aggregate of about $80,000,000 on alterations and improvements made in connection with the new passenger and freight terminals. —Editor.

In the structural design of this roof the common arrangement of trusses radiating from a central hub was found impracticable: (1) no provision could be made for a continuous circular tie for the radial members owing to the shape of the interior space, partly circular and partly rectangular, with the center point of the radial members in an open space; (2) the restraining of considerable horizontal forward deflection at the crown transversely through the top of the high barrel-arch truss, entirely open below, presented a problem with largely indeterminate factors: it was not considered feasible to limit this by means of base anchorage of the arched radial members at the elevated position of their springing lines in the building; (3) the plan of the perimeter of the dome at the concourse floor level does not conform with any symmetrical arrangement of columns supporting radial ribs.

After various alternative plans had been considered the structural design was fixed as a series of transverse, parallel, barrel-arch trusses (not acting as arches), dividing the dome into segments. This is the normal treatment for the combined structure, being standard for the barrel arch or front portion

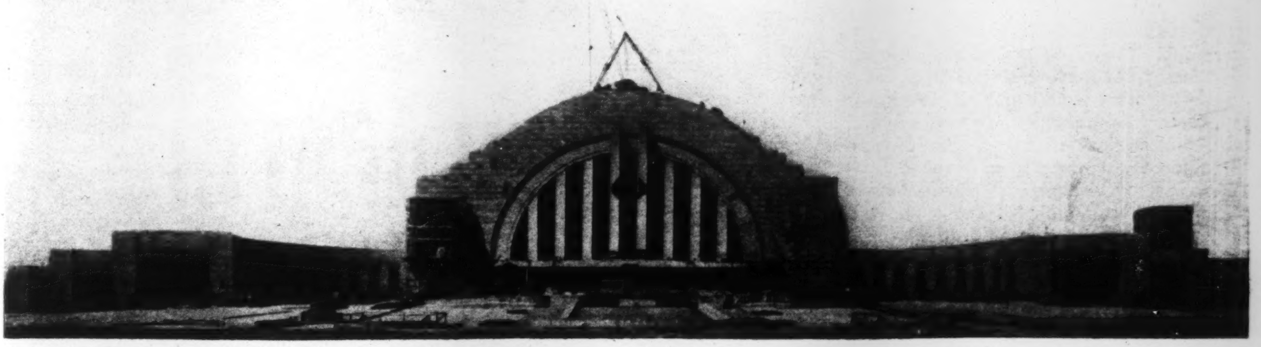


Fig. 1—Cincinnati union station fronts on inclined approach. At right and left are entrances to three loops through the station for taxis, buses and street cars.

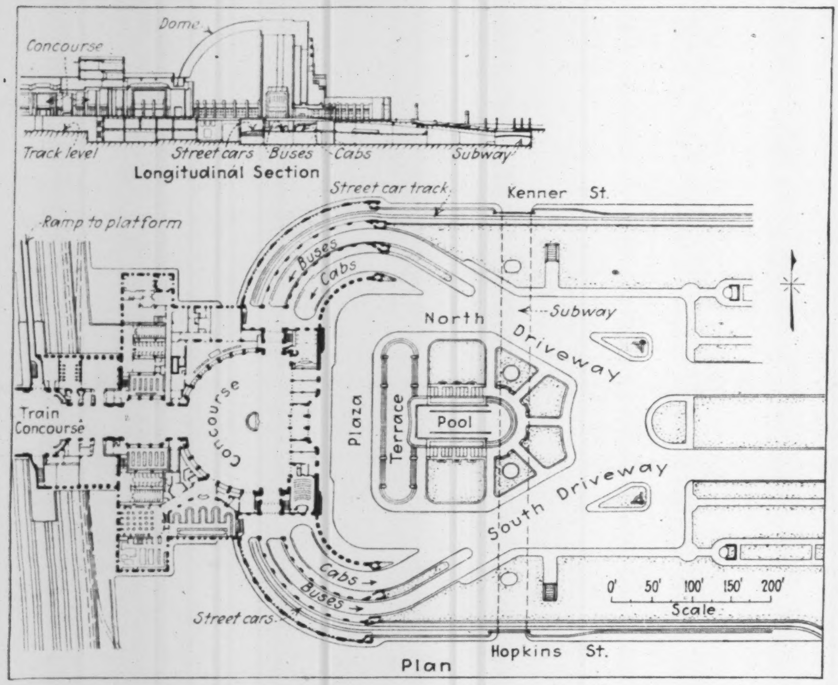


Fig. 2—Plaza approach to Cincinnati union station includes a park and provides for separation of vehicle traffic.

and well adapted to the semi-dome at the rear.

Unusual problems were presented in the working out of this structural design. Arch construction could not be used. The structural springing line is some 90 ft. above the ground floor, and the maximum external span is about 202 ft. for the dome and 209 ft. for the barrel arch. This is believed to be the largest half dome on any permanent building in the world. Except for the front portion, there is no opportunity for the necessary abutments or buttresses; and even in the front portion the required restraint for arches of this magnitude would seriously restrict the interior spaces of the building. This is evident from the drawing of the dome framing (Figs. 4 and 5).

Dome truss design

For the above reasons the main roof members were ultimately designed as rigid trusses. In the preliminary design of the maximum truss, based on the normal 18,000-lb. maximum fiber stress, it was found that the lateral deflection under final total loading would be about 1 ft. While this would present no serious problem in a bridge, it was obviously impracticable in a building. Restraint of this lateral movement by means exterior to the trusses led to results somewhat similar to those above mentioned in connection with the arch type and were likewise impractical of application in the unbraced ring at the rear of the dome.

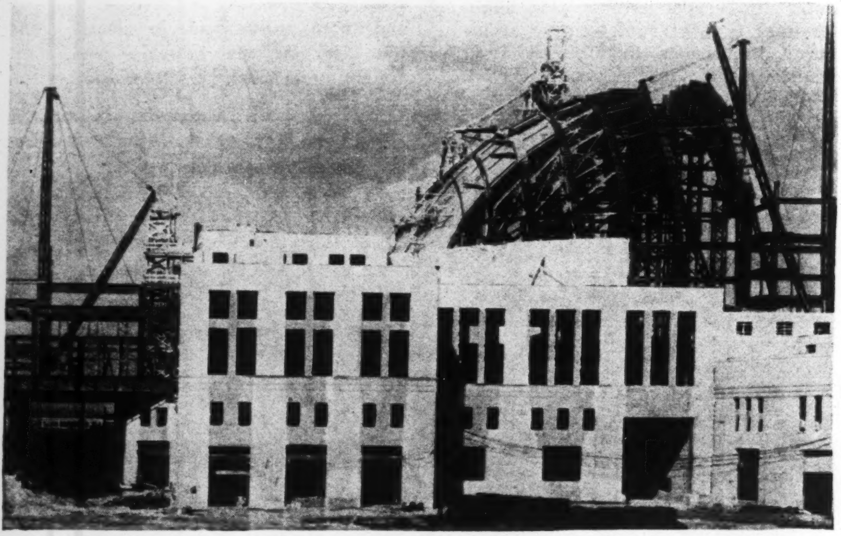
The reactive force was accordingly supplied in the truss itself, increasing its sections to the point where the maximum permissible thrust movement was attained and confining the lateral displacement within the predetermined limit. At truss 3 (Fig. 5), the largest segment of the dome, this permissible movement was fixed at $3\frac{7}{8}$ in., an amount estimated to be within the limit of danger of cracking masonry applied to the truss. It was further calculated that when the truss elongated this amount it would have sufficient rigidity to resist any further movement requiring consideration. To permit this stretch, all trusses were erected from fixed pin points at their north ends. Where the buttress support from the adjoining framing was insufficient to guarantee the immovability of the north pin of the truss, additional temporary anchorage by means of steel cables and turnbuckles was employed.

Framing of dome

There are eight of these trusses, with spans decreasing from 209 ft. for No. 1 to 67 ft. 8 in. for No. 8 (Fig. 4). Their spacing also decreases from 27 ft. for the two larger front trusses to 11 ft. at the rear. In the front trusses of the dome the chords are of built-up trough section (truss 3, Fig. 5), receiving the ends of the H-beam web members, so that no connection plates are required. In the two rear trusses the chords and web members are of H-beams, with connection plates (truss 7, Fig. 5). At their ends the trusses are built into the steel framing of the four-story flanking buildings, the top chord connection at one end being made with a pin in the top of a main column, except that no pin connection was needed for the short end truss, No. 8, which is of 67 ft. 8-in. span. Purlins and ceiling beams connect the trusses, the purlins being inclined to keep within the curve of the roof. Diagonal bracing is fitted between the top and bottom chords.

Upon the roof framing is a 5-in. concrete slab reinforced with wire mesh. This is covered with a three-ply felt waterproofing and then with ribbed terra cotta tile. The slab was poured in circular sections, beginning at the top. Each section was poured in both directions from the longitudinal center line, one end not being allowed to advance more than 10 ft. beyond the other. In this way the trusses were loaded equally. This same practice was followed with the domed plaster ceiling beneath the trusses. Live load, for wind, is taken as 25 lb. per sq.ft. of horizontal projection. Dead load comprises 25 lb.

Fig. 3—Erecting parallel arched trusses of dome on movable steel towers as falsework. Each completed truss was supported on towers until the next following truss was also completed.



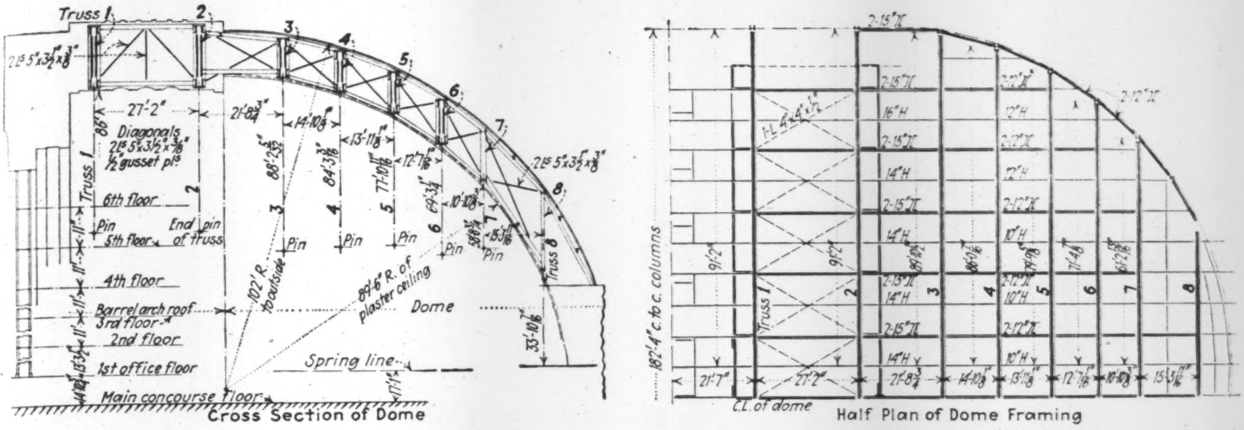

Truss 1
2
3
4
5
6
7
8
27'-2"
21'-8¾"
14'-10⅛"
13'-11⅛"
12'-7⅛"
Diagonals 2Ls 5x3½x⅜" ½" gusset pl's
2Ls 5x3½x⅜"
6th floor
5th floor
End pin of truss
Pin
4th floor
Barrel arch roof
3rd floor
2nd floor
1st office floor
Main concourse floor
102' R. to outside
89'-6" R. of plaster ceiling
Dome
Spring line
Cross Section of Dome
2-15"][
16" H
14" H
12" H
10" H
2-12"][
182'-4" c. to c. columns
21'-7"
C.L. of dome
Half Plan of Dome Framing

Fig. 4—Layout of steel framing for semi-dome roof of Cincinnati station.

for the tile and 50 lb. for the concrete slab. Transverse radius of roof ranges from 104 ft. 5 in. at truss No. 1, and 97 ft. 6 in. at No. 4, to 59 ft. 6 in. at No. 8; with corresponding ceiling radii of 86 ft., 84 ft. 3 in. (No. 4), and 33 ft. 10 in. (No. 8). Longitudinal radius of the dome is 102 ft. for the roof and 89 ft. 6 in. for the ceiling.

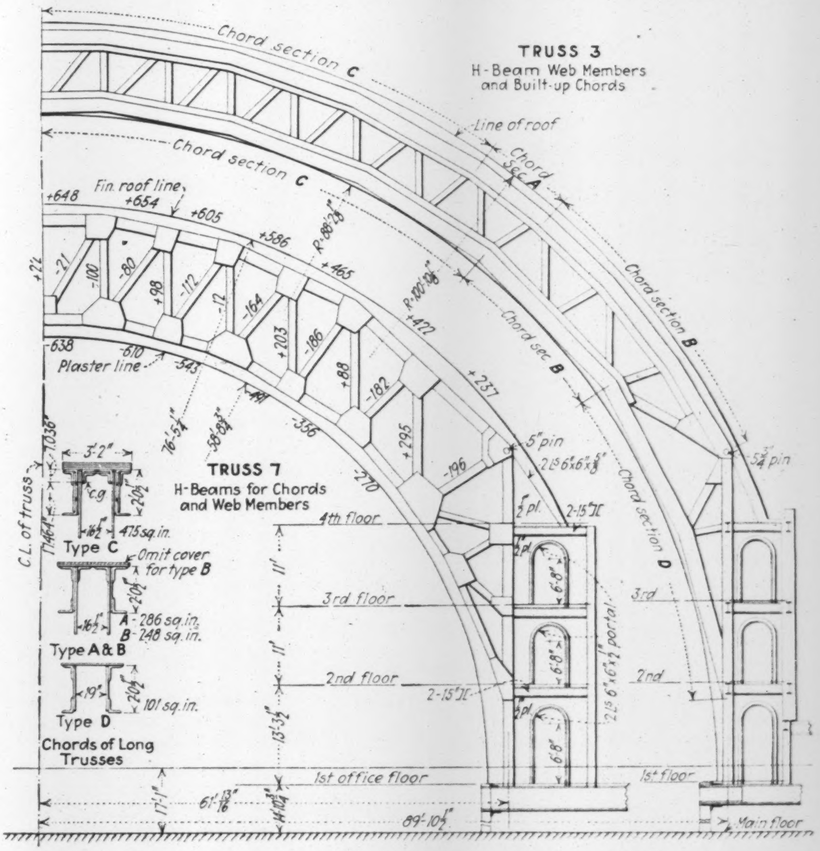
Erection of dome framing

The trusses were erected in sequence, beginning with No. 8 at the lowest and smallest segment of the dome. As this truss is substantially a part of the general framing of the building, it required no pin bearing or deflection adjustment as was necessary in the others. Portable steel towers surmounted by jacks were employed to support the trusses during erection, as shown under the right-hand truss in Fig. 3. (See also *ENR*, Feb. 4, 1932, p. 190.) Two towers were used for the smallest truss, and the number was increased with the spans until five were used under the largest trusses. They were positioned to handle best the unit parts as fabricated, lined, plumbed and securely guyed. Throughout the erection the full complement of towers was always maintained under the truss being erected and its immediate predecessor. The jacks were never released under a completed truss until the next following truss was also completed, together with the roof and ceiling tiers and all intermediate bracing members.

Each truss was fabricated short by the amount of calculated horizontal displacement due to its dead weight. The tops of the supporting columns on the south ends (opposite the pin bearings at the north ends) were leaned out of plumb northerly to permit the connections of the shortened truss. This point of flexure of these columns is some 40 ft. below the pin, being at the first office floor, which is the seat or origin of the columns for the dome section. The columns for the barrel arch (Fig. 4) extend about 50 ft. farther down, to the ground floor. The flexing of these longer columns was accomplished at a moment splice near the first office floor tier. All rear connections to the south columns at ends of trusses were temporary and were adjusted upon the release of the jacks under each truss, thereby permitting the columns to move southward toward their final plumb position. When further loads were later applied, the final connection holes registered and were then riveted.

Technically, the trusses end at the pin level, approximately at the fifth floor. The downward extension of the bottom chords to the columns at about the second-floor level furnishes the necessary keying in with the surrounding building structure and provides full restraint against any further lateral deflection due to weight of finish or future live loads. Progressive surveys were made of the completed work, and it was found that both crown deflections and lateral movements followed closely those anticipated in the calculations. One precaution in the design to provide for erection corrections proved unnecessary in practice. All roof and ceiling beams and the diagonal bracing between trusses were detailed with tolerance so that, if necessary, the trusses could be leaned

Fig. 5—Arched trusses are composed largely of H-beams, although heavy built-up chord sections are required for the higher trusses.


TRUSS 3
H-Beam Web Members and Built-up Chords
Chord section C
Line of roof
Chord sec. A
Chord section B
Chord sec. B
Chord section D
Fin. roof line
Plaster line
5" pin
5¾" pin
2Ls 6x6x⅞"
2-15"][
4th floor
3rd floor
2nd floor
1st office floor
1st floor
Main floor
TRUSS 7
H-Beams for Chords and Web Members
C.L. of truss
3'-2"
Type C
475 sq. in.
Omit cover for type B
Type A & B
A-286 sq. in.
B-248 sq. in.
Type D
101 sq. in.
Chords of Long Trusses
89'-10½"

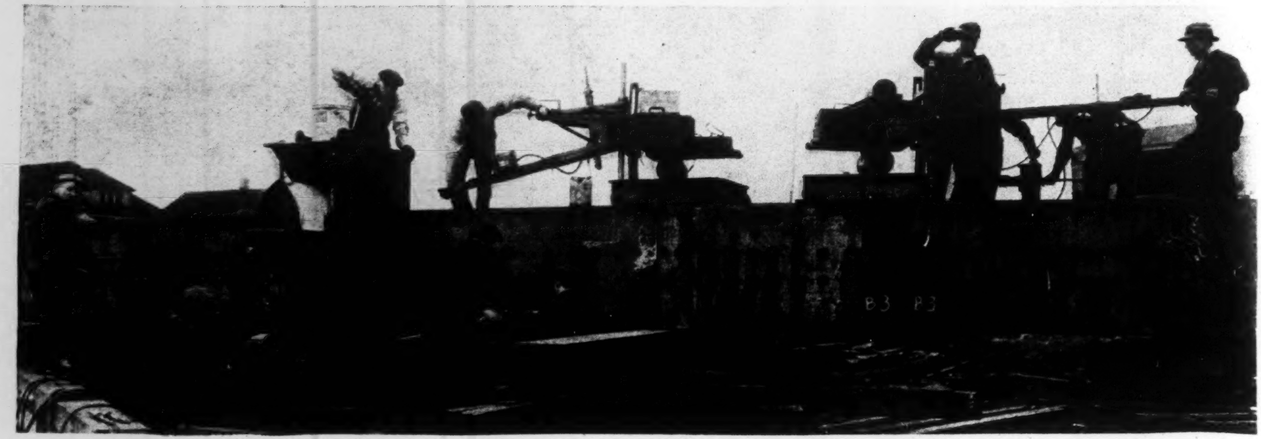


Fig. 6—Special power tools were devised by the fabricator for reaming heavy chord members.

slightly back of plumb in order that the natural forward push due to crown deflection might leave them accurately plumb upon release of the jacks. As the erection progressed and each truss unit was rigidly riveted to its connections with its predecessor, it was found that without further adjustments all subsequent trusses on erection were finally substantially parallel with the first pin truss, No. 7 (Fig. 5).

As the out-of-plumb amounts remained within allowable limits for the height of the vertical members, no further correction or adjustment was found necessary until the last truss, No. 1, was completed. The final adjustment to bring the front wall structure plumb was then made in the front-panel connections between truss No. 1 and the wall. As indicative of the uniformity of the load reaction on the completed structure, no cracks have developed in the large monolithic concrete envelope covering the entire roof area since the building was completed.

These trusses, the heaviest weighing 380 tons, were fabricated in sections and then assembled and reamed in the fabricator's yard before shipment, to insure accurate alignment of rivet holes. Reaming the holes in the deep and heavy truss sections was done by radial drills and special power-driven reamers. In the chord at the maximum crown section the 1¼-in. rivets have 7-in. grip. In Fig. 6, a section of built-up top chord, laid on its side, is having the rivet holes in the web reamed by machines riding on the chord. At the left a special traveling reamer is reaming the holes in the coverplates.

The main columns that support the trusses are composed of 14-in. H-beams with three coverplates, 16x1 in. to each flange. The component parts of the columns were tightly assembled and then spot-welded before the rivet holes were drilled.

Designers and builders

The structural designs for this station were prepared for the Cincinnati Union Terminal Co. (H. M. Waite, chief engineer) by the engineering department of Fellheimer & Wagner, architects and engineers, New York City, under the supervision of the writer. The shop details and fabrication of structural steel were done by the R. C. Mahon Co., Detroit, Mich., and the erection was handled by the Bass Construction Co., Cleveland, Ohio, as subcontractors for the James Stewart Corp., of New York, general contractors for the station building.

Sewage Disposal Trends in New York Metropolitan Region

INCREASINGLY rapid progress has been made in the last few years in the development of district or joint sewage projects in the New York region, according to Geo. W. Fuller, who discussed the subject at the recent joint meeting of the Central States Sewage Association and the Illinois Association of Sanitary Districts at Rockford, Ill. District bonds have not been issued; funds have come from county, intercounty, municipal or inter-municipal sources. An interstate commission has been formed on formal legal lines, the Tri-State Compact, which, however, is still to be ratified by two of the states and by Congress. Rental or service charges for sewers are now legal, made so by recent New York and New Jersey legislation. These charges, which may cover operation, maintenance and at least part of capital carrying charges, are assuming increasing importance in municipal finance of sewage works in the region.

Dilution is still the prevailing method of disposal, and great improvements have been made in the use of submerged multiple outlets far removed from shore. Fine screens are favored where there are long deep outfalls for the discharge of a highly chlorinated effluent and where they are assumed to be followed later by activated-sludge plants.

Sedimentation in newer installations is accomplished chiefly by single-story tanks with the sludge removed by mechanical means. Digestion commonly takes place in separate sludge-digestion tanks provided with covers and arranged for the collection and utilization of the gas. Contact beds and sprinkling filters are rarely if ever proposed for new projects in this region. Sand beds for filtration find their place chiefly in small plants on streams tributary to domestic water supplies and where state authorities have required so-called complete treatment together with chlorination to be followed by sand beds.

Sludge from the sedimentation tanks at the Passaic Valley plant and at New Haven is barged to sea, and the same step has been provided for part of the sludge from the Wards Island plant now under construction. Vacuum filters for dewatering sludge are receiving active consideration for several projects in this region, although conversion of the filter cake into fertilizer is meeting unfavorable commercial conditions. Incineration of dewatered sludge is meeting with more and more general favor.

Sewage-disposal betterments are still being retarded in this region by the requirement of the public health laws that state authorities have to prove *actual* injury rather than *potential* menace to the public health and property. This situation spells delay either through procrastination or the formalities of litigation in courts of original jurisdiction and appeals from the decisions thereof.

Providing an Elevator for Carlsbad Cavern Tourists

Shaft through 754 ft. of rock makes national park cave easy of access by means of high-speed elevator, which eliminates tortuous trail formerly used

By Frank A. Kittredge and Walter G. Attwell

Chief Engineer and Associate Engineer, Respectively, National Park Service

TO ELIMINATE a 750-ft. climb that formed an irritating but necessary part of a visit to the major features of the Carlsbad Cavern, Carlsbad Caverns National Park, N. M., the National Park Service has provided a passenger elevator by which tourists are taken to the main level of the cave. The elevator installation will make this natural wonder accessible to a much larger proportion of the thousands of visitors who visit the park. During the year 1931 nearly 100,000 tourists visited the cavern, but many of them were prevented from seeing the underground features because of physical inability to climb the trail back to the surface. The elevator will make directly available 5½ miles of level or slightly rolling trail leading to the main points of interest. The development consisted of providing a high-speed passenger elevator of modern design and safety equipment in a shaft drilled simultaneously from top and bottom through 754 ft. of rock to the main cavern level.

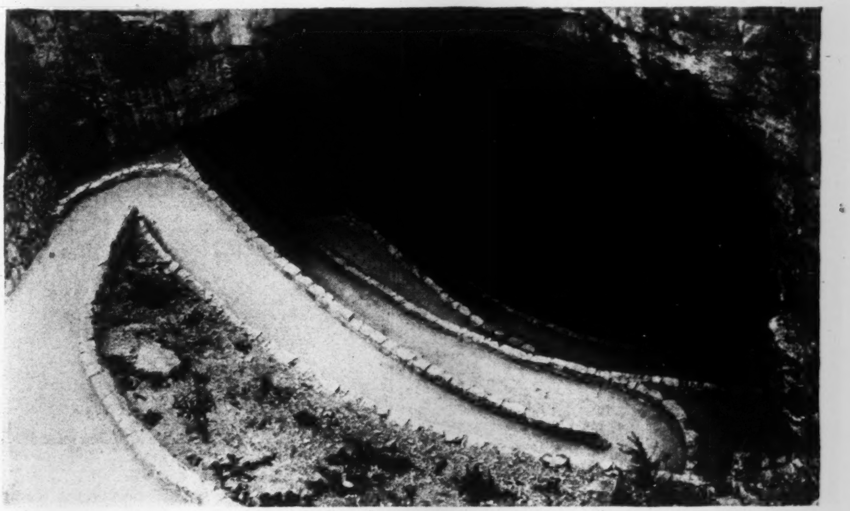
Cavern

The cavern in this national park, located about 26 miles from Carlsbad, N. M., contains the largest series of underground caverns ever explored, extending through 50 miles of known chambers and corridors. It is quite certain that miles of unexplored caverns lie near the elevator shaft, but they have as yet been undeveloped. In general, the rooms and formations are larger but similar to those in other limestone caverns, including fantastic designs as well as the usual stalactites and stalagmites. One of the major features is the Big Room near the lower entrance to the elevator, which is 4,000 ft. long and has a maximum width of 625 ft. and an average ceiling height of 200 ft. This room contains 57 acres of floor area and could comfortably seat 500,000 people, or the entire population of the state. Another formation is the Giant Dome, 62 ft. high and 18 ft. in diameter containing a striking resemblance to the leaning tower of Pisa. Many of the more interesting formations are made visible and have their beauty enhanced by a system of indirect lighting.

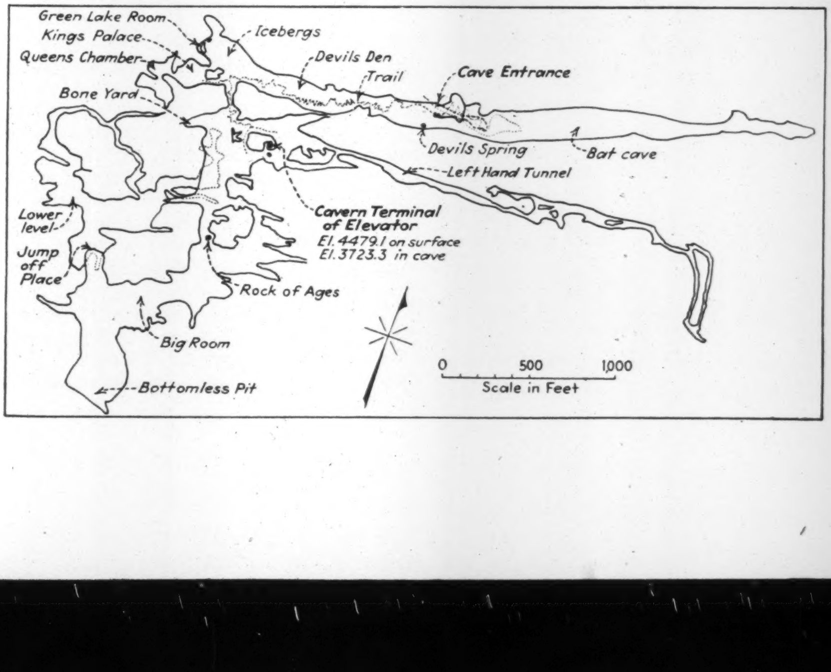
Survey

Location of the elevator shaft was dictated by the necessity for making the lower end accessible to the main trail and major formations. Selection of this point and the desirability of sinking and raising the shaft at the same time necessitated an accurate survey up a circuitous trail to the surface. This survey was run in three independent traverses, with methods similar to usual mine-survey procedure.

The traverse line started at the base of the elevator-shaft location and extended over cliffs through dark passageways and tunnels to the mouth and then to the position of the top of the shaft. Before starting instrument work, permanent brass monuments were concreted into the solid rock at angle points and benchmarks. The engineering party rotated positions on the different traverses to lessen the chance for repetition of error. Angles were turned four times with an ordinary mining transit, and verticle angles were read to the minute. All distances were chained on the slope to the nearest hundredth of a foot. Slope distances as measured were corrected for sag and a constant factor of temperature, since the air in the cave remains unchanged at 56 deg. throughout the year. Outside the temperatures were about 20 deg. when the surveys were made in January, and corresponding corrections were made. Completion of the shaft indicated a closing error of 0.02 ft. horizontally and 0.01 ft. vertically.

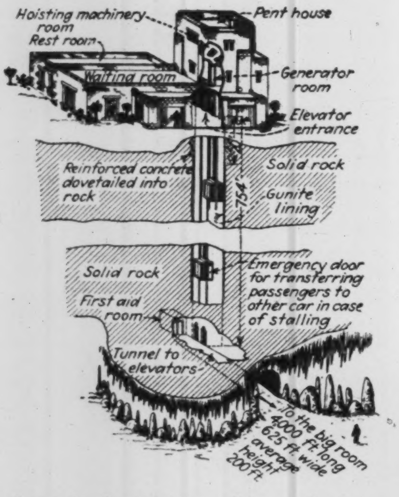


Entrance to trail into cavern. The elevator now makes possible the trip for all those who before were physically unable to make the return climb.



Map of Carlsbad Cavern, showing location of elevator in respect to principal points of interest.

Working three shifts a day from top and bottom, driving operations were carried on without an accident and met 450 ft. below the surface. The shaft size was 6 ft. 10 in. x 14 ft. 3 in. inside the concrete lining. Blasting rounds consisted of 36 holes taking out an average lift of 5 ft. The 4,000 cu.yd. of limestone excavated were removed with an average of 6 lb. of explosive per cubic yard without injuring any of the formations in the adjoining rooms. The rock consisted of limestone in horizontal planes and was badly eroded. Small passageways a few inches in diameter caused much steel to be broken and many shots ineffective. This feature made drilling slow and costly.


Hoisting machinery room
Rest room
Pent house
Waiting room
Generator room
Elevator entrance
Reinforced concrete dovetailed into rock
Solid rock
Gunite lining
754'
Solid rock
Emergency door for transferring passengers to other car in case of stalling
First aid room
Tunnel to elevators
To the big room 4000 ft. long 625 ft. wide average height 200 ft.

Isometric of new elevator layout and concession and office building in Carlsbad Caverns National Park in New Mexico.

Shaft lining

The shaft was lined with gunite varying in thickness from ½ in. to several inches. The lining is principally to prevent weathering with possible displacement of loosened fragments. The gunite was mixed in proportions of one of cement to three parts of sand, with 4 per cent of calcium chloride added to hasten the set. High-early-strength cement was used for the walls and for concreting the steel I-beams in place to permit the resumption of drilling below an hour later and because of the blasting operations about 100 ft. below.

The walls were washed with water under 125-lb. pressure immediately ahead of the guniting. The gunite was mixed at the surface and shot vertically a maximum distance of 750 ft. to the nozzles under pressure varying from 40 to 125 lb. No segregation of material was noted, and rebound averaged about 20 per cent. Measurement of the quantity of gunite was made from the materials used, deducting the 20 per cent of rebound. It was found impracticable to measure the volume of material on the surface because of irregularities. The elevator, house, shaft and guide construction are entirely fireproof, with guide-rail supports of I-beam sections set directly into the rock walls.

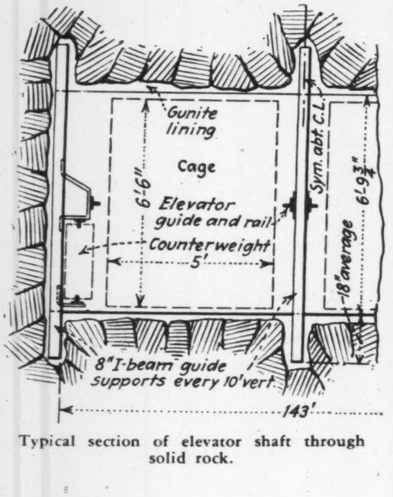
The steel headframe is the center of a three-story shaft house. The first floor is occupied by the elevator landing, the second floor is used as a generating room, and the third contains the machinery and controls. The roof will be used as an observation gallery, providing an unobstructed view for a distance of more than 200 miles to the south. The shaft house will be surrounded with a building containing lobbies, concessions, lunch rooms and office. This building will be of Indian Pueblo type of architecture and will be constructed during the year.

A novel feature of this building will be the provision for natural heating, cooling and ventilation, using air from the cavern, which is always at a temperature of 56 deg.

The elevator is of modern electric type with self-leveling cage carrying twelve persons. It is planned to ope ate at a speed of 700 ft. per minu One elevator has been installed at th present time, and a second will added later in the other compartmen With the exception of a few of th elevators in the Empire State Buildin the present installation is the longe single-lift passenger elevator in th world. The equipment contains a automatic safety devices found in usual building installations.

The cost of the double shaft and on cage was $88,292. The shaft-excava tion contract was held by Charles H Dunning, Phoenix, Ariz. The elevato machinery was furnished by the Pacifi Elevator Equipment Co., San Francisco. The work was carried out under the direction of the authors, and the installation of the elevator and electrical machinery was under the supervision of Ira S. Stinson, assistant engineer, National Park Service. Horace M. Albright is director of the National Park Service, and Thomas Boles is superintendent of Carlsbad Caverns National Park.

Typical cavern scene made available to all by new elevator installation.



Typical section of elevator shaft through solid rock.

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Properties of Fiber Boards

Wall boards and insulating boards, the two principal classes of fiber building boards, have been studied at the Bureau of Standards. Tests were made on fifteen samples of wall boards and eight samples of insulating boards representative of the commercial products in common usage. Weight, thickness, density, strength, absorption of moisture, and changes in dimension with changes in moisture content were studied. There was no marked difference in the strength of the different boards studied. The average thickness of wall boards was less than that of insulating boards, but the density of the wall boards was greater, and the strength was approximately the same for both classes. There were no marked differences as regards absorption of moisture or changes in dimension with changes in moisture content, except that the crosswise expansion of the laminated or built-up boards was greater than that of homogeneous or one-piece boards. The difference in thermal conductivity between the various insulating boards has been found by previous work at the bureau to be small, and the average wall boards are so thin and of such density as to be considered of relatively much smaller value as heat insulators. The results of laboratory tests of the various boards are tabulated, the test methods used are described, and the different properties considered important are discussed in detail in Bureau of Standards *Miscellaneous Publication 132*, purchasable from the superintendent of documents, Washington, D. C.

Investing Unemployment Relief in Civic Improvements

By H. C. McClintock

Director of Public Service, Boulder, Colo.

UNEMPLOYMENT relief in Boulder, Colo., was confronted in the spring of 1931 with the charter tax limit reached and the inadvisability of creating a general-obligation bond issue. The Boulder Employment Council of five citizens was organized to secure relief funds by popular contributions. This body handled all money, card-indexed applicants for work, kept the accounts, paid the workers and acted as an employment agency for citizens having temporary employment to offer. There was no overhead cost, all accounting and clerical work being done by the unemployed. The only municipal aid was free use of tools and equipment. The unemployed listed numbered 500. Employment was alternated on various schedules, and the rate of pay to all was $2.50 a day. Details of management were very carefully worked out but are not particularized, so as to allow emphasis in the article of the more important fact that all employment was for city improvement. —EDITOR.

EARLY in the unemployment relief work it was decided that any projects undertaken should be worthwhile, permanent public improvements for the benefit of the city at large. Since, of necessity, this relief work must be carried on during the winter months, suitable projects were hard to find. However, with suggestions offered by the Planning and Parks Commission and the various city departments, the projects undertaken have produced improvements of such a nature that the citizens have received full value for the money they have contributed.

Water supply improved

Some improvements to the city water system have been planned for the spring months of 1932. This program was set ahead several months and started in the dead of winter. First, some 700 ft. of new 4-in. water main was installed, so that men could be working while a larger project was outlined and organized. This project consisted of removing 2,100 ft. of 4- and 6-in. water main and replacing it with 8-in. cast-iron pipe. The pipe thus removed was then disjointed and reclaimed for future use. Next, 600 ft. of 6-in. and 225 ft. of 2-in. mains were installed. This was followed by lowering 1,600 ft. of 6-in. main a depth of 3 ft. Since these projects were undertaken in cold weather, with all the difficulties attendant thereto, and since the crew was made up of inexperienced men and the personnel was changed every three days, the Employment Council paid for half the labor.

Buildings repaired

The public library is built on a clay hillside, which is more or less saturated with seepage from an irrigation ditch. Due to this fact and also to insufficient footings, the front wall had settled, pulling away from the other walls and causing unsightly cracks in the brickwork. The repair of this building was undertaken with unemployed labor. Substantial braces were placed against the outside of the wall, and underpinning was placed under the footings. Excavation was then made to a depth of 4 ft., and a new concrete footing 8 ft. wide and 40 ft. long was poured. After the concrete had set and the braces and underpinning had been removed, the brick and stonework was repointed.

Boulder is located at the very edge of the foothills of the Rocky Mountains and owns thousands of acres of mountain park lands in the immediate vicinity of the city. At one location on this land a stone shelter house had been erected in 1924. This building has been in need of repairs and alterations for some time, and as there were some stone masons on the list of unemployed who were willing to work at their trade at the prevailing wage, they were put to work on this building. The fireplace, which did not work satisfactorily, was taken out and a new one built. The stone wall was removed from the south end of the building and the north end was closed, thus affording a wonderful view to the south end and providing protection from the north. This shelter house has been quite inaccessible to motorists, so a new road, leading to the building, was constructed by hand with pick and shovel. A well some 20 ft. in depth was dug to the south of the house, lined, and a pump was installed, providing an amply supply of water for picnickers.

Boulder Creek improved

Boulder Creek flows through the center of the city from the west to east city limits. It is a source of potential beauty. For fifteen years the Planning and Parks Commission has been purchasing the property along this stream with the view of beautifying it. With unemployed labor the creek banks have been cleaned for a distance of three city blocks, the underbrush has been cut out, and riprap walls have been built along each bank for neatness of appearance and protection from erosion. This spring a systematic plan for planting native and specimen trees and shrubs was carried out.

This creek, like all mountain streams, is subject to flood periods, and although no serious floods have occurred since 1894, flood protection has been receiving consideration for many years. At one point in the west part of the city a sand pit was constructed many years ago, necessitating an abrupt bend in the creek bed and producing a decided flood menace. A few years ago this property was purchased by the city to alleviate this condition. Having the labor furnished by the Employment Council this winter, the work was put under way. It was not practicable to do all the work by pick and shovel, but as several of the men who registered with the council owned teams, they were hired at the rate of $5 per day for man and team. With this kind of a crew a channel 40 ft. wide was cut through the upper and lower banks of the pit, and a dam was built across the old channel, providing a straight course for the stream. Since we did not wish to lose the scenic value of the pond formed by the pit, a headgate of concrete and 3-in. plank is being constructed in the lower channel to retain the pond. In case of flood the plank may be removed, permitting an unobstructed flow of the stream. The banks of the new channel were riprapped with stone, and incidentally an unsightly house foundation was razed and the material used for riprapping.

Recreation projects

Beauty was added to that of nature in Bluebell Canyon south of the city in the mountain parks. Two dams were built in the canyon, and thus two small lakes were created. The natural growth of trees, plants and shrubs will be augmented with additional planting in conformity with a plan furnished by S. R. DeBoer, consultant landscape architect for the city. Trails have been constructed along the shores of these lakes leading into a beautiful rugged mountain district.

A shelter house has long been desired at the summit of Flagstaff Mountain, another section of city park land. One crew of men at this location removed large stones from the roads and parking areas and piled them at the site of the proposed shelter house, to be ready for use when construction is undertaken.

Boulder is proud of her ownership of the best night soft-ball diamond in the state of Colorado, which was opened last summer. The sport proved very popular, the bleachers providing seating capacity for only a small minorty of the spectators. Consequently this winter another section of bleachers with seating capacity for 350 people was erected. The new and old bleachers and the fence along the base lines were painted.

Organization

With 40 to 60 men working on these various projects, it was necessary to create several crews. Foremen for each of the crews were selected from our regular street and water department forces, their wages being paid by the city. The work was directed by J. J. Morse, water superintendent, Charles Gustafson, street superintendent, and John Mills, park superintendent, under the supervision of the author as director of public service.

Design of Highway Guard-Rail Dictated by Tests

Results of tests of four types of guard-rail—Motion-picture camera an invaluable test aid—Tests point to a definite design theory and structural type—Other safeguards often superior to guard-rail

By Searcy B. Slack

Bridge Engineer, Georgia State Highway Board, Atlanta, Ga.

THE EARLY conception of the guard-rail on highway embankments was that it served chiefly the purpose of a warning sign. Textbooks of comparatively recent date speak primarily of its psychological value and stress the importance of its high visibility. These are still elements of the guard-rail problem, but the thought of highway designers is fast turning away from consideration of the guard-rail primarily or even in moderate degree as a danger sign. Its essential purpose is held to be moderation of accident hazard, first by preventing overrunning of the embankment by a wild car, and second by guiding the colliding car back onto the traveled path with minimum shock to the car and the occupants. This is the design conception presented so excellently in the article that follows. It will be observed that the test results and design conclusions lead directly away from much modern guard-rail construction practice. They therefore invite wide consideration and comment by highway designers. —EDITOR.

RELIABLE information as to the behavior of the various types of guard-rails under impact is rather meager. Comparatively few tests have been made, and reports from accidents are so scanty of detail and conflicting that it is difficult to draw conclusions. To secure first-hand information, field tests on several types of guard-rails were undertaken by the state highway board of Georgia.

Description of tests

The tests made in Georgia were conducted by the construction of a runway about 400 ft. long on a 9 per cent grade built in such a way that vehicles released along the runway would be guided into a section of the guard-rail to be tested. By releasing the vehicles at different points along the runway it was possible to vary the speeds of the vehicles, at the time of impact, from 15 to 25 miles per hour. The vehicles used were touring cars weighing 3,500 lb., a truck weighing 14,000 lb., and a bus weighing 10,800 lb. empty or 16,300 lb. loaded.

The guard-rails to be tested were built across the lower end of the runway at known angles with the center line of the runway. The earth in which the posts supporting the guard-rail were set was a stiff red clay containing 50 to 60 per cent sand, and it gave excellent support to the posts. The posts were spaced so that the center line of the runway passed midway between two posts. The angle between the runway and the guard-rail was either 20 or 40 deg., and tests on all types were made at both angles. The types of guard-rail tested were:

1. Woven wire and expanded metal fencing. In this group were included 2x2-in. mesh wire, 6-gage, 2x4-in. mesh wire, 4-gage, 2x5-in. expanded metal fencing, and 6-gage 2x2-in. mesh wire, with an 18-gage deflecting plate 8 in. wide fastened along the lower edge of the wire. The posts for this group were of 6x8-in. pine timber, 7 ft. long, set $3\frac{3}{4}$ ft. in the ground.

2. A low wooden rail consisting of a 4x8-in. creosoted timber bolted to 6x8x6-ft. wooden posts spaced 10 ft. apart set 4 ft. in the ground. To the face of the wooden rail was bolted a 16-gage steel plate.

3. Two $\frac{3}{4}$-in. steel cables fastened by J-bolts to 6x8-in.x7-ft. pine posts set 4 ft. in the ground and 10 ft. apart. The cables were anchored securely at each end to heavy timber deadmen. The cables were offset from the posts about 3 in. by wooden blocks.

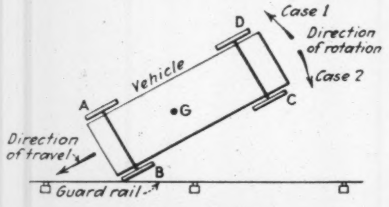


Fig. 1—Diagram illustrating action of forces when motor vehicle strikes guard-rail at angle.

4. Steel-plate-type rail. This rail consisted of a 16-gage plate 12 in. wide fastened by 3-leaf cup-shape springs to 6x8-in.x6-ft. wooden posts spaced 16 ft. apart, and the springs were fastened to the posts with $\frac{3}{4}$-in. bolts.

The speed of the vehicles at the time of the impact was determined by timing with a stopwatch the travel over the last 50 ft. before impact. The speed was also checked by noting the number of frames of moving-picture film required for the vehicle to pass a row of stakes set at 5-ft. intervals parallel to the runway. The time interval between the frames of the moving-picture camera was checked by taking a picture of a clock with a large second hand. By counting the number of frames required for a given travel distance the speed of the vehicle could be estimated.

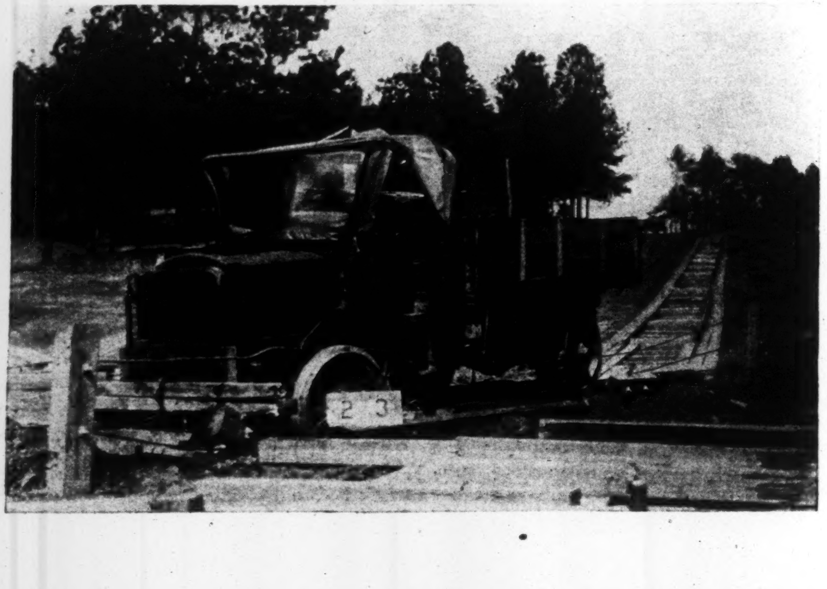
Moving pictures of the tests were made by two cameras, one of which was placed at right angles to the direction of travel, and the other placed behind the guard-rail. The best information on what happened during the tests was obtained by a careful study of the moving pictures.

Summary of results

Based on the results secured from 22 impacts, the following general conclusions as to the design of the guard-rail were drawn:

1. Wooden posts 6x8-in., set 4 to 5 ft. in the ground, give excellent support to the guard-rail. Only two posts were broken in the 22 tests that were run. There is apparently little difference between the resistance of the posts set with the 6- or the 8-in. side toward the roadway.

Fig. 2—Runway and cable-type guard-rail after impact by truck. Note how truck knocked down two wooden posts and tore free the J-bolts fastening the cables. One wheel passed under the upper cable.



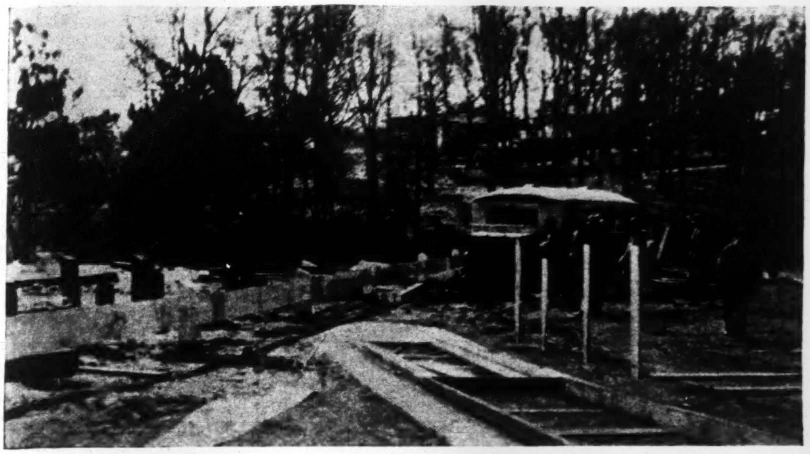


Fig. 3—Plate-type guard-rail after impact by bus. Note how the bus was deflected.

2. To be effective, the rail between the posts must be sufficiently strong to resist the impact and to transmit the force of the impact to several posts. Unless several posts act together they are not sufficiently strong to deflect the vehicle. The rail should be fastened to the posts securely and should be offset from the posts to allow a vehicle to glide by without striking the post.

3. A type of rail that will deflect the vehicle, without entangling, as well as transmit force to several posts, seems to be the most effective. The deflecting element should be fairly smooth and free from projections that would damage the vehicle gliding along the rail.

Design problem

The ideal guard-rail would be a highly visible rail, not prone to cause snowdrifts, which would in an emergency deflect a vehicle back into the road with a minimum of damage to the vehicle and rail. Note that the suggested function of the guard-rail is to deflect the vehicle rather than stop or check the speed of the vehicle. If a guard-rail should stop a vehicle going at a high rate of speed within a distance of 1 or 2 ft., the damage to the vehicle and the passengers would be about the same as if they went over the embankment. In addition to this, such a tremendous strength would be required to stop or check the kinetic energy of the fast-moving vehicle that it would be impracticable to build a rail to meet such a requirement.

The vehicle consists, essentially, of a load supported on four wheels. The diameter of the wheels varies from 24 in. to as much as 40 in. for the larger trucks. The wheels are designed to carry the load and to check the speed of the vehicle, and they are securely fastened to the chassis through the springs. The guard-rail would, therefore, function best against the wheels or bumper of the vehicle and would do least damage to these parts. Any part of the guard-rail striking above the wheels would not be effective in deflecting the vehicle until it had good bearing against the body, hood or radiator of the vehicle, all of which are rather easily crushed.

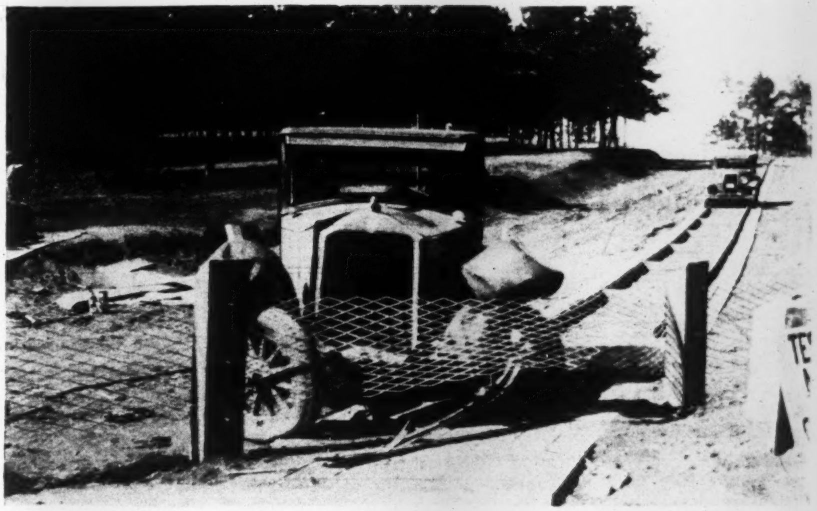


Fig. 4—Mesh-type guard-rail after impact by touring car. Note how back end of touring car swung around, heading the car against the post.

Fig. 1 shows a vehicle supported on four wheels striking a guard-rail. The wheels are represented by letters *A*, *B*, *C* and *D*. The center of gravity of the vehicle is at some point *G*. If wheel *B* strikes the guard-rail and the guard-rail tends to slow up the speed of the vehicle, a moment is developed tending to head the car more into the guard-rail. The moment will be represented by the amount wheel *B* is retarded by the guard-rail multiplied by the distance from the center of gravity of the vehicle to the guard-rail. The direction of rotation would be as shown by the arrow in case 1. On the other hand, if wheel *B* is not retarded by the guard-rail but glides along the face of the rail, the reaction tends to turn the car parallel to the guard-rail, and the direction of rotation of the car would be in the direction of the arrow in case 2.

From the above it will be seen that a guard-rail that will not entangle the vehicle but will permit it to slide freely along the face of the rail will give the best results. The height of the guard-rail should be such that it would best engage the wheels of the vehicle. If the lower edge or element of the rail is placed about 13 in. above the ground, and the upper edge or element of the rail is placed from 24 to 27 in. above the

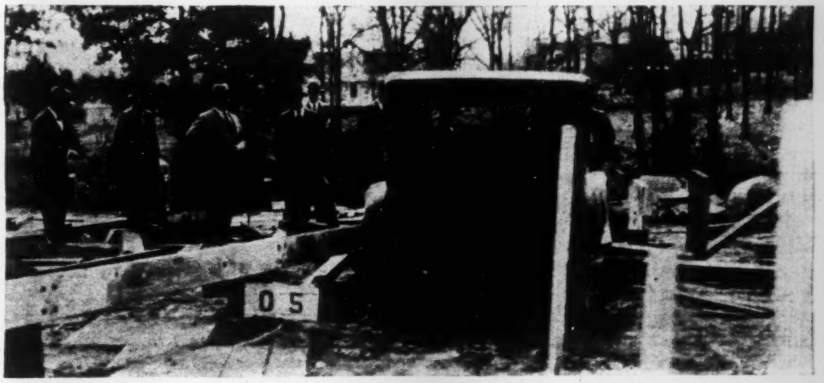


Fig. 5—Low wooden guard-rail with metal faceplate after impact by car. Note how car was deflected without any damage to the guard-rail.

ground, the rail would engage the wheels to best advantage.

Support for guard-rail

The support for the guard-rail should preferably be resilient and strong enough to withstand reasonable shock from a vehicle passing along the guard-rail. Concrete posts, although reasonably permanent and highly visible, are not resilient; and if struck by a vehicle, they would either break at the ground line or cause considerable damage to the vehicle. Wooden posts are resilient, and when struck by a vehicle they will frequently be knocked entirely out of the ground by the vehicle without great damage to the posts or the vehicle. The tests that were made indicated that 6x8-in. wooden posts set 4 to 5 ft. in the ground afford excellent support to the rail.

Either wooden posts should be of a very durable species of timber or they should be treated to prevent decay. Several types of steel posts are available, but the author has not first-hand information on this kind of post.

The distance between the posts will vary with the type of rail. The spacings used in the sections tested were 10 ft. for the wooden rail and cable and 16 ft. for the plate-type rail. The tests indicated that these spacings were satisfactory.

Rail element

The rail between the posts has been the subject of much controversy, but it would appear that several types of construction can be used effectively.

If wooden rail is used, it should consist of fairly heavy timber, not less than 4x8 in. or 4x10 in. in section, and the face of the wooden rail should be protected by a metal plate that is fastened at frequent intervals to the rail by flat or round-headed lag screws or carriage bolts. The metal plate will prevent the hub or bumper of the vehicle from digging into the rail, and it affords a smooth gliding surface for the vehicle. The plate gives tensile strength to the rail and will serve to hold broken ends of the rail together. There are numerous accounts of accidents with wooden guard-rail in which the rail has been broken and has been forced into the vehicle, with fatal results in many cases. If the wooden rail is bolted at close intervals to the steel plate, it does not seem likely that such an accident would occur.

There are several types of rail on the market in which the rail element consists of a steel plate. This type of rail should prove very effective provided the steel plate is strong enough and is securely fastened to the posts, preferably through a spring element. The plates should be offset from the posts by the springs or other means, so that the vehicle can glide by a post without striking it. Too much emphasis cannot be placed on working out details of fastenings that will really develop the strength of the steel rail element. Tests should be made on these fastenings and connections, as the computed strength of connections in thin metal is uncertain. This is the defect in most designs of this kind.

In the usual construction of cable-type rail one of the most common errors in design has been the height at which the cables were placed. The usual design requires the cables to be placed 15 to 18 in. above the ground and from 15 to 18 in. apart. This places the cables rather far apart, and one of the cables is above the center of gravity of the average vehicle. If the lower cable were placed 12 to 13 in. above the ground, and the upper cable placed 12 to 13 in. above the lower cable, the cables would be placed so that the lower one would be just a little below the hub of the average vehicle, while the upper one would be below the top of the wheel. The least damage would be done the vehicle if the cables were spaced so that they would not strike any part of the vehicle except the wheels and fenders. Also maximum strength from the cables would be expected, as they would be engaged on the same part of the vehicle and would act together. With the cables spaced widely apart, the lower cable generally strikes the wheel just above the hub, while the wheel passes under the upper cable, and it must engage some other part of the vehicle.

The usual fastening used on cable-type rail is a J-bolt that fastens the cable to the posts, generally through a cast-iron or some other type of block serving to offset the cable from the posts. In this construction the strength of the cable is not developed by the fastening to the posts. The J-bolt will develop only a strength of from 800 to 2,000 lb. (depending upon how tightly it is drawn up), and this does not represent the strength of either the post or the cable. The J-bolt projects beyond the face of the cable, and if a vehicle glides by the J-bolt, it knocks out the bolt or a part of the vehicle. A better type of fastening would be a spring or a block that would offset the cable 4 to 5 in. from the post and would cover only the back two-thirds of the cable but would grip the cable so as fully to develop the strength of the cable in the fastening. This would leave a smooth surface on which the vehicle could glide by the posts and would put the posts into longitudinal as well as lateral play. Too much emphasis has been placed on the end anchorage of cables, and too little attention has been given to fastening effectively the cables to intermediate posts. Because of their closeness to the point of impact the intermediate posts must generally exert the force to deflect the vehicle.

The discussion in this paper has been limited to three of the simpler and more generally used effective types of rail. The same underlying principles of design may be applied to any type of rai

Maintenance

From the standpoint of maintenanc the rail element of the guard-rail shoul show a surface that can be easil cleaned and painted and should be o such construction that a light impac from a vehicle would do no damage t the rail other than scrape off the paint. In the types of rails that are made up o bent, cupped or corrugated plates th rails are easily bent out of shape b light impact and cannot be readil straightened so as to present a neat appearance. This necessitates replacin an entire panel of rail, even though th rail was only slightly damaged. Th cable-type, simple wooden rail and metal face plate or flat steel plate afford easily painted surfaces and are of such construction that they will not be easily damaged by a light impact, and even though bent they can be straightened in the field.

Limitations of use

Guard-rail is essentially a provision for safety, and any discussion would not be complete unless attention were called to the objections to guard-rail and to other types of construction that perhaps afford equal safety under many conditions.

Among the objections to the use of guard-rail are that it adds to the maintenance cost and prevents maintenance of the shoulders by machinery. It also prevents mowing weeds or cutting grass out to the shoulder line with a mowing machine. The guard-rail restricts the use of shoulders for parking or traffic purposes in cases of emergency.

In many places the slopes can be flattened or the width of shoulders can be increased so as to provide additional safety for traffic without the use of guard-rail. For example: at the current prices for earth embankment and for any of the usual types of guard-rail the slopes of an embankment 8 ft. high could be flattened from 1 on 1½ to 1 on 3 somewhat more cheaply than a guard-rail could be built. The width of the shoulders on the 8-ft. fill could be increased by 7 ft. somewhat more cheaply than guard-rail could be constructed. Either flattening the slopes or increasing the width of the shoulders is much safer construction than guard-rail in a great many places.

Summary

Guard-rail should be of such construction that it will deflect vehicles by allowing the vehicle to glide along the face of the rail.

The height of the rail should be such that it will effectively engage the wheels of the vehicles.

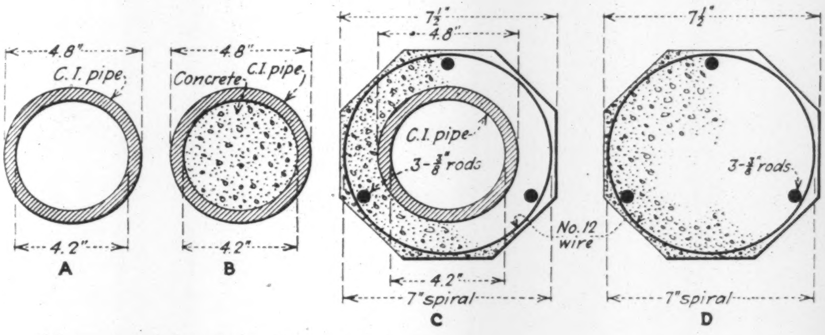
Consideration should be given to other methods of protecting traffic at dangerous sections of the highway.

Cast-Iron Pipe for Columns and Column Cores

Test of centrifugally cast pipe as columns and as hollow cores in reinforced-concrete columns 42 to 105 in. long show high strength

RECENT TESTS of columns of cast-iron pipe made by the centrifugal process are said to be the first tests of cast-iron columns in this country for more than thirty years. They were made to verify the claim that such pipe is specially suitable for columns because of the density of metal and the uniformity of wall thickness. Of twenty pipes, ten were tested as columns and ten as cores in reinforced-concrete columns.

All the pipes were 4.2 in. in inside diameter and 0.3 in. thick, with a sectional area of 4.3 sq.in. Eight columns (*A* in the accompanying table and drawing) were of pipe alone, 56 to 105 in. long; two (*B*) were of pipe filled with concrete; and ten (*C*) were of hooped concrete with the pipe as a core. For comparison, two solid reinforced-concrete columns (*D*) were tested. Only the longer columns are included in the table, except for the solid column and a comparable cored column. The cored and solid concrete columns were octagonal in section, 7½ in across opposite sides, with three ⅜-in. vertical bars and a spiral hooping of No. 12 wire. The pipe, bars and spiral represented 21.1, 1.9 and 1 per cent, respectively, of the cross-sectional area within the hooping. Gravel concrete of $1:1\frac{1}{2}:2\frac{1}{4}$ mix was used, with 6 gal. of water per bag of cement. The columns were 23 days old when tested.



Columns tested for comparative strength. *A*, cast-iron pipe; *B*, pipe filled with concrete; *C*, hooped concrete with pipe core; *D*, hooped solid concrete.

To determine the load carried by the cast iron in the cored columns, the hooped concrete was assumed to have a strength of 7,000 lb. per sq.in., as stated in the report. Then the 20.4 sq.in. area of concrete carried 143,000 lb. plus 13,000 lb. for the three vertical bars, or a total of 156,000 lb. The difference between this load and the ultimate load was considered as the load carried by the core. This load for the pipe cores is about 50,000 lb. per sq.in., or slightly more than that for the pipe columns and agreeing closely with tests made by the U. S. Bureau of Standards, noted in *Engineering News-Record*, Feb. 13, 1930, p. 278. Total compression at ultimate load was about ¼ in. in the 70-in. pipe columns and $\frac{5}{16}$ in. in the cored columns.

For the plain cast-iron pipe columns, *A*, the ultimate loads ranged from 201,000 to 217,000 lb., or 46,800 to 50,300 lb. per sq.in., showing a high degree of uniformity. For the concrete-filled pipes, *B*, the ultimate load was 266,500 lb., as against 205,500 lb. for plain pipes of the same length. The solid concrete columns, *D*, showed high strength, failing at 285,000 and 324,000 lb. For the cored columns, *C*, the ultimate loads ranged from 338,000 to 400,000 lb. In only one case were the pipe and the spiral broken; this column had failed at 378,000 lb., but the load was continued until the spiral was burst and the column had 1½-in. deflection in its 84-in. length. The pipe was not broken transversely but had two splits for nearly its full length.

TESTS OF CAST-IRON AND CONCRETE COLUMNS

Column		Cross-Sec. Area		Spiral Reinf., Sq.In.	Vertical Reinf., Sq.In.	*L/R*	Ave. ult. load, 1,000 lb.	
Type	Length, In.	Cast Iron, Sq.In.	Concrete, Sq.In.				Total	Per Sq.In. on Cast Iron
A	70	4.3	None	....	....	44	213.5	49.6
A	84	4.3	None	....	....	52	201.5	46.8
A	105	4.3	None	....	....	66	205.5	47.7
B	105	4.3	13.8	....	....	66	266.5	47.7
						L/D		
C	70	4.3	20.4	0.385	0.33	10	368.0	49.2
C	84	4.3	20.4	0.385	0.33	12	372.5	50.3
C	105	4.3	20.4	0.385	0.33	15	353.0	49.9
C	42	4.3	20.4	0.385	0.33	6	369.0	49.5
D	42	None	38.5	0.385	0.33	6	304.5	....

***A*, cast-iron pipe columns. *B*, pipe columns with concrete filling. *C*, hooped concrete columns with cast-iron pipe cores. *D*, solid-hooped concrete column.**

These tests were made at the Armour Institute of Technology, Chicago, under the direction of Prof. Huntley, for James B. Clow & Sons, manufacturers of the pipe. L. J. Mensch, civil engineer, Chicago, planned the series of tests and supervised the construction and testing of the columns.

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Straits of Gibraltar Tunnel Scheme Revived

The longest submarine tunnel in the world, connecting the continent of Europe and Africa at the Straits of Gibraltar, is once again being projected, this time with the backing of the new government of Spain. The shortest possible distance between the two shores at the Gibraltar Straits is more than 6½ miles. This would mean that the tunnel itself would have to be nearly 7 miles long at a minimum and also entirely under water.

The agitation in favor of a tunnel beneath the Straits of Gibraltar began many years ago, when French and Spanish capitalists saw the advantages that such a project would give their nations. Spain is interested because the tunnel will bring her closer in contact with her African colonies. France is interested because of the railroad tunnel would shorten the distance between Paris and South America. Passengers could travel from Paris to West Africa in three days, and in the future cross to South America by airplane, shortening greatly the time now required by the fastest boats.

As early as 1920 a committee was formed in Barcelona to study the possibilities of the Gibraltar tunnel. In 1929 the Spanish government appropriated 3,500,000 pesetas (about $500,000 then) for preliminary surveys. Under Lieut.-Col. Pedro Jovenois and a corps of engineers the depths of the strait were sounded, preliminary rock borings made, and a possible tunnel was mapped out between Tarifa, Spain, and Alcazar, Spanish Morocco. Work was abandoned in January, 1930, when the original appropriation was expended. Since that time nothing has been done.

Suspension Bridge Stresses Determined by Model

Application of model study at Princeton University to the Mt. Hope bridge gave results so close to analytical design values that model analysis was adopted for preliminary and check design of the Trans-Bay bridge at San Francisco

By George E. Beggs, Elmer K. Timby and Blair Birdsall
Princeton University, Princeton, N. J.

MODEL STUDY of suspension bridges, untried three years ago, has already proved invaluable during development of designs for the projected crossing between San Francisco and Oakland in California. It was also used in solving some of the erection problems on the Hudson River bridge at New York. These applications were based upon procedure developed at Princeton University, using a model of the Mount Hope bridge. In the following article the Mount Hope bridge-model studies are described in detail. The San Francisco bridge tests followed the same procedure, except that wind loads were simulated, while only dead and live loads were considered on the Mount Hope model. This article then may be considered as presenting the science of suspension-bridge model testing as it exists today. —EDITOR.

MECHANICAL solution of the live-load and temperature stresses and deflections of a suspension bridge by means of a steel model was first carried out, so far as the authors are aware, in the spring of 1930 in the engineering laboratories of Princeton University. The Mount Hope bridge in Rhode Island, built during the preceding year, was chosen as an example, inasmuch as very complete design results, arrived at by using Moisseiff's deflection theory, were available from D. B. Steinman, the designer. The results obtained from the mechanical solution not only led to the conclusion that models of proposed suspension bridges may be used for the predetermination of stresses due to live load, wind and temperature, but also revealed that the usual theory (which assumes all suspender stresses to be equal for any condition of live load and temperature and assumes zero elongation of the suspenders) gives results for cable stress and bending moments in the stiffening truss that are slightly in error.

Following the successful conclusion of the experiments at Princeton, the first job application of suspension-bridge model analysis was carried out last summer for the engineers of the California state highway department, to assist them in selecting a design for a suspension bridge between San Francisco and Goat Island as a part of the Trans-Bay bridge to Oakland. The work was carried out at Berkeley under the direction of one of the authors (Professor Beggs) and Prof. R. E. Davis, of the University of California. Test procedure, scale of model and results sought were similar to those in the Mount Hope bridge model studies. On the other hand, the San Francisco models were larger, and multiple-span as well as single-span arrangements were examined; there were also minor differences in the make-up of the models, occasioned by differences in the design of the two bridges.

Because the Mount Hope bridge tests were basic, and as they differ only in detail and not at all in degree of accuracy of results from the more recent Trans-Bay bridge tests, this article will be confined largely to the earlier experiments at Princeton. With the exception of brief reference to the scope of the Trans-Bay bridge model tests (*ENR*, July 16, 1931, p. 113, and Sept. 10, 1931, p. 428), this article represents the first published account of this development, introduced in 1930 and since subjected to some refinement.

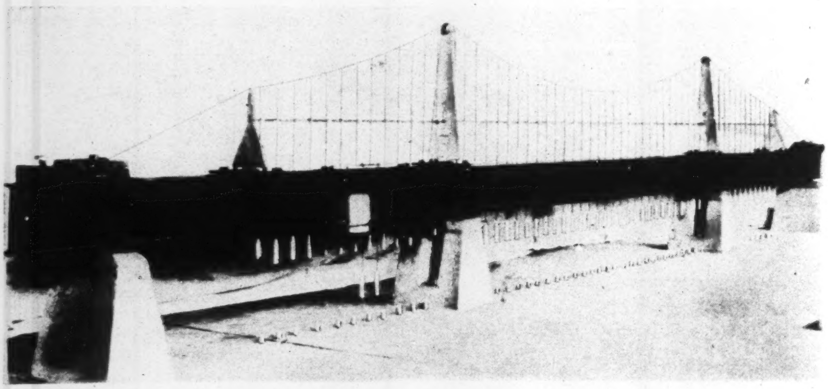

Fig. 1—Model of 1,200-ft. span Mount Hope suspension bridge, with dead-load weights attached to the suspenders and live-load weights resting on the floor. Towers and cable bents are made of steel plates, mounted on two cone points. Temperature effects are simulated by moving the anchorages horizontally.

Mount Hope bridge

The Mount Hope bridge, for which the model was constructed, was described in *Engineering News-Record*, April 12, 1928. It has a 1,200-ft. suspended main span, 504-ft. suspended side spans, and a roadway, including sidewalks, about 32 ft. wide. The cable sag in the main span is 120 ft. The assumed loading for the floor system was three lines of 20-ton trucks spaced 33 ft. on centers, plus 30 to 37 per cent impact; for the trusses, three lines of 15-ton trucks spaced 33 ft. on centers, and for the cables, 750 lb. per linear foot, plus 2,700 lb. dead load. The steel towers, 284 ft. high above mean low water, are of the fixed-base flexible type and each consists of two eight-compartment cellular posts. The stiffening trusses are 18 ft. deep and discontinuous at the towers. Expansion in the stiffening trusses is provided for by rocker connections on the towers.

Principles of model analysis

The problem was to design and construct a loaded model from which the stresses, strains and deflections of the full-sized structure might be predicted. Inasmuch as the magnitude of the deflections of a suspension bridge creates a new system of equilibrium for each condition of loading and temperature, it was realized at the outset that the model, deflected by loads or temperature, must be geometrically similar to the structure under corresponding conditions. In other words, the model and the structure must have the same unit strains. It remained to derive a simple formula for calculating the model dimensions and loads in terms of the corresponding values for the structure and the linear-scale reduction factor, designated by the letter n.

Live-Load Stresses—If any particular member of the structure has a cross-section A and is subjected to a total stress S, due to loads P on this structure, the area of the corresponding model member will be A/n^2, and the model loads must be reduced to P/n^2.

Temperature Stresses—One method of accounting for the temperature strains, stresses and deflections of the large structure caused by a *rise* in temperature is to state that the structure is deformed and stressed in its effort to become larger than the foundation system supporting it. It is evident, as commonly assumed in developing theoretical formulas for temperature effect,

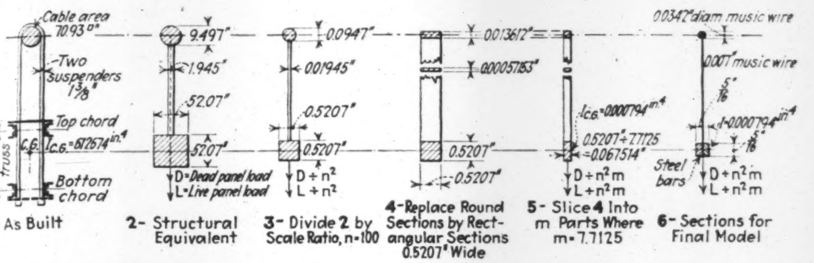

Fig. 2—Steps followed in proportioning a suspension bridge model. Two reduction factors, *n* and *m*, are used, the first equal to 100 (in this case), and the second *m*, so chosen that a commercially available size of music wire results. The same factors are applied to the actual bridge loads.

that strains, stresses and deflections equivalent to a temperature rise would result by reducing the size of the foundation system in every direction in such proportion as would conceivably occur if a steel foundation system contracted freely with falling temperature, while the supported steel structure remained at a constant temperature.

To produce (for rise of temperature) the equivalent unit strains and stresses and geometrically similar deflections in the model as in the structure, it is only necessary to move each foundation support of the model in a straight line (horizontal or inclined) toward a common reference point by an amount equal to *ctl*, where *c* equals the coefficient of expansion (for steel), *t* equals degrees rise (60 deg. F.) in temperature, and *l* equals the straight-line distance from any model foundation support to the selected point of reference. For simulating fall of temperature the procedure is reversed.

Deflections—In the above method the deflected model is always geometrically similar to the structure under the corresponding conditions of load and temperature. Therefore, to obtain the corresponding deflections for the structure, it is only necessary to multiply the measured deflections of the model by the linear-scale reduction factor *n*.

Proportioning of model and loads

The steps followed in proportioning the different parts of the model are set forth in Fig. 2. Sketch 1 shows the cross-section of one cable, suspenders and stiffening truss as built. The final model to be proportioned and loaded is for loads in the vertical plane only, although it may be noted that the same methods can be extended to include wind loads.

In sketch 2 of Fig. 2 the structural equivalent of the cross-section shown in sketch 1 is given, to which is applied a panel load of such amount that its weight plus one panel of this equivalent structure equals the dead panel load of the structure as built. The live panel load applicable to the structural equivalent would be the same as for the structure as built.

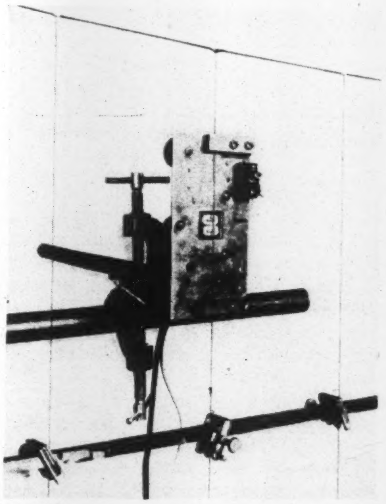

Fig. 3—Musical tensometer used to determine variation of suspender pull. Design of instrument is based upon fact that a given gage length of wire changes its pitch as its stress changes. Model cable is shown above the instrument, while the bar below is the model stiffening truss.

In the usual case, where Young's modulus *E* for any model member differs somewhat from the known or assumed *E* for the corresponding member of the structure, proper compensation is made in designing the model member by decreasing its section when its *E* is larger than that of the member of the structure, and vice versa, so that corresponding unit strains remain the same.

Arbitrarily and for reasons of convenience, a linear reduction scale *n* equal to 100 was chosen for proportioning the Mount Hope model. The application of this linear reduction factor to sketch 2 gives the linear dimensions of the model shown in sketch 3, and dividing the actual loads by the square of the reduction factor gives the model loads.

It was found that the model loads, D/n^2 and L/n^2, were inconveniently large for handling, and it was believed that a solid cable 0.09497 in. in diameter was too stiff for convenient use. Further reduction was necessary. The development of the reduced sections is set forth in sketches 4, 5 and 6 of Fig. 2. In sketch 4 ribbons of rectangular section replace the round cable and suspender shown in sketch 3. The loads in sketch 4 would be the same as in sketch 3. Sketch 5 shows the section of a model that would result from dividing the sections and loads in sketch 4 by the factor *m*, the value of which is so chosen that the areas of the cable and suspender would be very nearly equal to the commercial standard music wire shown in sketch 6. For convenience in model assembly the stiffening bar in sketch 5 is replaced in sketch 6 by two stiffening bars. In Fig. 1 the loads D/n^2m are seen hanging below the bridge model, and the weights corresponding to the live panel loads, L/n^2m, rest on the floor beneath the model.

Construction of the model

Foundations—Concrete piers were constructed in position below the main towers and anchorages. Upon these piers two 12-in. channels were supported, being fixed to one end pier and provided with expansion rollers at the other three piers to insure that this steel foundation would expand and contract at the same rate as the model (whenever temperature variations occurred within the laboratory) without producing temperature stresses within the model. Nevertheless, care was taken to maintain even temperature, because small parts of a model respond more quickly to temperature change than heavier sections. The method of simulating temperature stresses and deflections involved the movement of

DEFLECTIONS AND HORIZONTAL CABLE STRESS, MOUNT HOPE BRIDGE

Condition of Loading	*H* in Kips	Cable-Bent Deflection at Intersection of Tangents (Inches)	Main Tower Deflection at Intersection of Tangents (Inches)	Truss Deflection at Center of Span (Inches)
1. Main span and far side span fully loaded. Temp. +60° F.	+867.3 (+872.0)	+3.12 (+3.21)	+14.19 (+12.76)	....
2. Same as (1) except Temp. —60° F.	+1,038.0 (+1,022.0)	+0.50 (+0.92)	+7.88 (+7.46)	....
3. Near side span fully loaded Temp. +60° F.	—36.7 (—33.8)	+1.44 (+1.37)	—3.50 (—3.62)	33.6 and (33.0) max. loaded side span
4. Same as (3) except Temp. —60° F.	+114.6 (+109.1)	—1.25 (—0.98)	—9.75 (—10.20)	....
5. All spans fully loaded Temp. +60° F.	+905.8 (+899.3)	+3.25 (+3.34)	+7.56 (+7.30)	....
6. Same as (5) except Temp. —60° F.	+1,076.0 (+1,058.0)	+0.62 (+0.99)	+1.31 (+1.95)	....
7. Main span fully loaded Temp. +60° F.	....	....	....	73.2 and (68.7) max. loaded main span
8. Main span fully loaded Normal temperature	....	....	....	45.6 and (48.8) loaded main span

The values not inside the parentheses were computed by Robinson and Steinman, consulting engineers of New York City. The parenthetical values were obtained from the steel model.

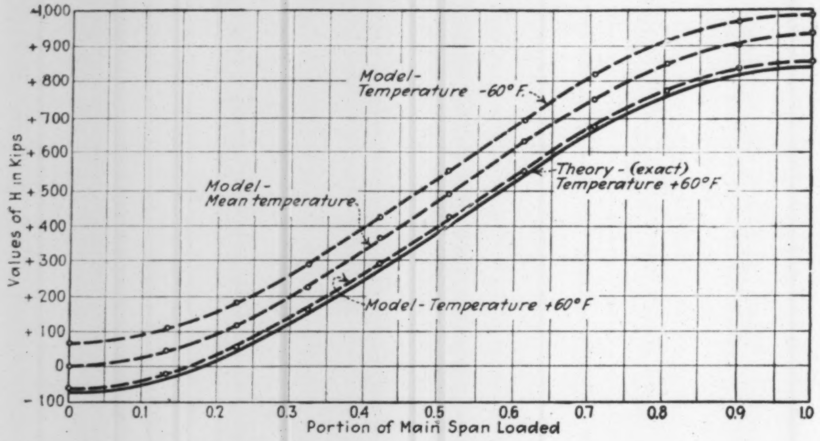

Fig. 4—Variation in horizontal component of cable stress, *H*, for uniform live load advancing on the main span of Mount Hope bridge. Note the close parallelism between theoretical and model results.

model supports under conditions of constant temperature, measurements being made with Ames dials mounted on the foundation structure and connected to the movable supports.

Main Towers—Stresses in the tower itself were not studied with this model. The resistance of a tower to load is best determined by using a larger steel model of the tower alone. (Tests made at Princeton within the last few weeks have shown this to be entirely practicable.) To reproduce the condition in the model of weightless rocker towers assumed in the design, the relatively heavy main steel towers were supported on two cone points and were counterweighted with a pendulum of such weight that the center of gravity of the tower and pendulum combination was located in the line of the two points of support. On the top of this rocker tower a special cylindrical piece of steel was placed so that a saddle support of correct proportion was provided. In case that it is desired to simulate fixed elastic towers, the counterweights must be increased to give the desired resistance to tower deflection, and the tower foundation must be moved when simulating temperature effects. This was the case with the Trans-Bay bridge models. If roller saddles must be simulated, the balanced tower as used for the Mount Hope model would be proper.

Cable Bent—In the theoretical design the cable bents were assumed to be weightless and with rocker supports. To simulate the rocker supports, two cone points were used at the base of each model bent. No counterweighting pendulum was used for the cable bent, for this tower was relatively light in weight, and its deflection from the vertical was at all times of small amount.

Cable Anchorage—The anchorage of the model cable was so designed as to permit horizontal motion. The vertical component of the cable stress was resisted by dead weight, and the horizontal component by adjusting screws. This movable support was provided to make it possible to simulate the effect of temperature, as has been previously explained. Vertical movement of the anchorage point, as theoretically required, was not provided for, as its effect was seen to be negligible for this bridge.

No provision was made for a corresponding displacement of the rocker tower bases, but their properly calculated movement was taken into account in predicting the tipping of the towers due to temperature changes. However, in a design considering the rigidity of towers fixed at the base, as in the Trans-Bay bridge, it is necessary that these bases be shifted when studying temperature effects.

The Cable—In planning the cable and the suspender connections a table was computed giving the distance from one end of the cable to each suspender connection and to each tower connection, on the assumption that all stress was removed from the cable. The cable and a standard steel tape were then stressed with spring balances until their unit strains were identical. Suspender points, tower and cable-bent reference points and anchorage points were then located at such intervals that when the cable was placed on its supports and dead load was applied the suspenders would occupy their correct positions. Loops were placed in the lower ends of the suspender wires, to which the weights representing the major part of the dead panel loads could be attached by their hooks. These dead-load weights had rings in the bottom from which the live-load panel weights could be hung.

The completely fabricated cable with its attached suspenders was then transferred to its position in the bridge, and the main portion of the dead load was attached to the suspenders. In addition, small temporary erection weights, not shown, each equivalent in weight to one panel of model stiffening truss and its connecting clamp, were hung beneath, so that the cable then took the shape corresponding exactly to the condition of full dead load and normal temperature. The correctness of the construction at this time was carefully checked by measuring the sag in the cable in the main and side spans, and the error in construction proved to be negligible.

Stiffening Truss—In agreement with the assumption made in the design of the structure, the stiffening truss must have zero bending moment under the condition of full dead load and normal temperature. This was accomplished by placing the two steel bars of the model stiffening truss on a special falsework built up from the two 12-in. channels. In choosing the elevation for the erection of the stiffening bars a position was taken somewhat lower than that corresponding to the structure itself, in order that experiments might be more easily carried out to determine the variation of live-load and temperature stresses in the suspenders by means of a special musical tensometer. It is believed that this departure from the true elevation of the stiffening truss had little effect on the experimental results for the reason that the elongation of the suspenders is of relatively little importance in its effect on stresses, strains and deflections. As a matter of fact, in all theoretical calculations the elongation of the suspenders is neglected.

The stiffening bars rested freely on the supporting falsework. Small C clamps, such as those used by chemists, were used to fasten them tightly to the suspender wires. The connections of these stiffening bars to the towers by pendulum expansion links (similar to those used in the actual structure) was also made at this time. The falsework was then removed, and the temporary erection weights corresponding to the weight of the stiffening bars and clamp were removed. This scheme evidently provided a stiffening truss in which there was no bending moment under the condition of full dead load and normal temperature in the model.

Using the model

Cable Stress—For finding the stress due to live load or temperature change in the model cable a 20-in. Anderson strain gage developed in the Bell Telephone laboratories was attached to the backstay at each end between the cable bent and the anchorage.

In order that the strain readings taken on the backstay might be interpreted into stress a simple method of calibration was employed, in which it was determined that one gage division (1/25 of the circumference) was equivalent to 86,555 lb. of horizontal component of cable stress in the full-size structure. It is believed that this dial permitted cable unit stress to be read within an accuracy of 55 lb. per sq.in.

Deflection Apparatus—The theoretical calculations had already indicated that the maximum deflection of the cable and stiffening truss due to live load and temperature approached 70 in. for the structure. The corresponding deflection for the model was therefore anticipated to be one-hundredth as great, or 0.70 in. It was therefore apparent that the model deflections could be measured with sufficient accuracy with a steel scale graduated to hundredths of an inch and observed with a reading glass.

Tower and Cable-Bent Deflections—The theoretical calculations had also indicated that the main tower and cable-bent deflections would be relatively very much smaller than the stiffening-truss deflections, thus requiring extremely accurate measurement. For this purpose a filar micrometer microscope capable of reading 0.0001 in. was supported from the foundation structure in such position as to observe the edge of a small ink dot placed on a target at the top of the tower to be observed. An accurate calibration of this microscope against a standard scale showed that one micrometer division was equivalent to 0.000198 in. It follows that one micrometer division of deflection measured on the model is equivalent to 0.0198 in. of deflection on the actual tower or cable bent of the structure.

Stiffening-Truss Bending Moments—It was decided that the best way to measure the bending moment in the stiffening truss was to observe its change in curvature. Preliminary calculations based upon the theoretical live-load and temperature bending moments for the structure indicated that this curvature of the stiffening truss caused by these bending moments would be very small and difficult to measure. Two knitting needles were soldered to the stiffening bars at a distance apart equal to a definite gage length of 3 in. Small corks were pushed on the ends of these needles. Small hat pins about 3 in. long were stuck through these corks horizontally and arranged so that the points of the pins nearly met at the center of the gage length. The gap between these points was located at a definite distance (8 in.) below the center of the bars. When positive bending moment was produced in the stiffening bars by live-load and temperature rise, these pins would move apart. The change in the size of the gap between them, measured in a horizontal direction, would be a measure of the angular change between the sections to which the needles were clamped. This in turn would be a measure of the bending moment. A microscope identical with the one used to measure tower deflections was used to measure the change in the gap between the pin points. The value of one division of the microscope, in foot-pounds of bending moment, was found by calibrating the bending-moment apparatus on a simple beam made identical in section with the stiffening truss of the model and having a part of its length subjected to a constant bending moment.

In order to determine experimentally the position of live load producing maximum positive bending moment in any section of the model stiffening truss under observation, the model was adjusted to the condition equivalent to highest temperature by slackening off the adjustment nuts at each anchorage by a precalculated amount. Thereupon live-load weights were added one at a time in the immediate neighborhood of the section where maximum positive bending moment was sought, and by trial the position of loads was found by observing the condition of loading that caused the greatest separation of the pin points supported from the stiffening bars. It was found that whenever the pin points of the control beam or of the model stiffening truss separated one micrometer division the corresponding change in bending moment was 0.01549 ft.-lb. Since model loads must be multiplied by n^2m to secure full-size structure loads, model bending moments must be multiplied by n^2m times n, or n^3m, to secure the bending moments in the full-size structure. It follows therefore that the above value of 0.01549 ft.-lb. must be multiplied by 100^3 times 7.7125, which amounts to 119,500 ft.-lb. of bending moment in the structure, corresponding to one micrometer division of separation of the pins that were attached to the stiffening bars of the model. This value of 119,500 ft.-lb. corresponds to approximately 1 per cent of the maximum positive bending moment in the stiffening truss of the main span of the structure, and accordingly this method of measuring bending moment was considered to be sufficiently accurate for practical purposes.

In the Mount Hope bridge model the stiffening truss was of constant section, whereas in the Trans-Bay bridge models the stiffening trusses were made variable in section to represent the changes in section of the truss of the structure.

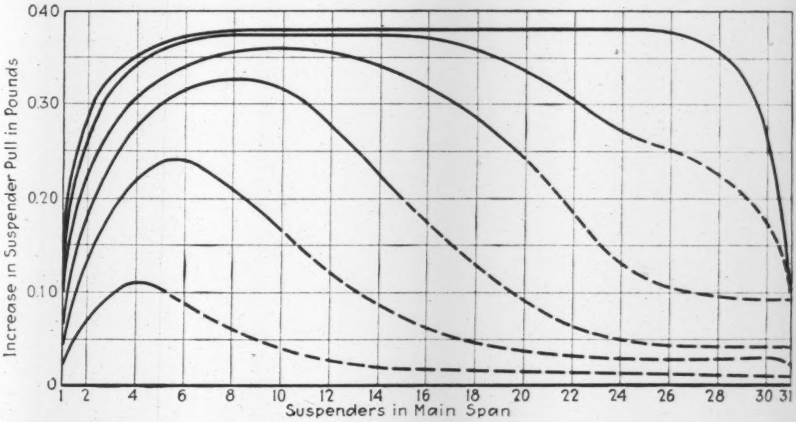

Fig. 5—Increase in suspender pull caused by advancing live load on main span, the temperature remaining normal. Deflection theory assumes suspender stresses to be equal and strains to be zero. Full lines in curves indicate extent of live load.

Suspender Stresses—Variation of suspender pull, caused by live load and temperature, was measured with a special musical tensometer developed at Princeton University by W. R. MacNamee under the direction of Prof. C. H. Willis. Stresses were found on the principle that a given wire on a given gage length changes its pitch (natural frequency of vibration) as its stress changes. The instrument developed had a definite gage length, an electromagnetic system of inducing a variable frequency of vibration and a telephone circuit for picking up the tone given off by the wire. A stress-frequency curve was found by testing several sample pieces of suspender wire. The frequency of the electromagnetic impulses was controlled with a dial on an oscillator. As this frequency approached the natural frequency of the stressed suspender, the wire vibrated more easily and therefore gave off more sound. The peak was found in the same manner that a station is dialed on a radio.

Summary of results

Horizontal Components—In Fig. 4 is shown the graph for *H*, the horizontal component of the cable stress, for uniform live load advancing on the main span. Also in the accompanying table are shown other comparative values of *H* for various conditions of loading and temperature. The experimental *H* curves drawn by dotted lines in Fig. 4 are each drawn exactly through the observed points for each curve. The smoothness of the curves obtained and their close parallelism indicate that the apparatus employed gave remarkably uniform results. Only one set of theoretical values was available for comparison, and these lie on the full line curve.

The difference between the experimental and theoretical *H* curves for the live load advancing in the main span is so uniformly of the same amount and sign as to lead to the belief that there is some definite cause for the small, though practically negligible, difference between the two. The writers believe that this difference arises from the basic assumption made in the deflection theory that the suspender stresses are equal and suspender strains are zero for all positions of live load, conditions that did not obtain in the model as shown by the musical tensometer measurements. Some values of suspender pull are shown in Fig. 5.

Bending moments

In Fig. 6 are shown the graphs for the maximum positive bending moments in the main and side spans as caused by combinations of live load and maximum temperatures. The experimental curve is shown by the dotted line plotted through the observed values indicated by small circles. This curve gives values of bending moment ranging about 10 per cent below those obtained by theoretical calculation. The writers feel confident that the experimental results obtained are close to the truth. They are of the opinion that this relatively unimportant difference between experiment and theory is due to the same cause that accounts for the difference between the experimental and theoretical H curves—namely, that the assumption is made in the theory that all suspender stresses are equal and that the strains are zero.

Conclusions

From the comparison of the theoretical and experimental results that were obtained for the stresses and deformations of the Mount Hope bridge, it appears that the deflection theory, as perfected by L. S. Moisseiff, agrees closely enough with experiment for satisfactory application in practice. Wherever a small difference between theory and experiment exists, as in the case of the cable stress and bending moments, the writers believe that this is due to the assumption in the theory that all suspender stresses are equal for any condition of live load and temperature and that the elongation of the suspenders is zero.

The success with which the Mount Hope model checked with careful theoretical analysis leads to the conclusion that models of proposed suspension bridges may be used with confidence in their design, not only for the determination of stresses and deflections due to live load and temperature but also due to other forces such as wind. In the special case where stresses and deflections due to wind are desired, the model wind-panel loads will equal the actual wind-panel loads W' for one-half transverse section of bridge, divided by n^2m. All parts of the model except stiffening truss are proportioned as before. A single model cable is used. In proportioning each rectangular section of the model stiffening truss in this case, the breadth and depth of the stiffening bar is calculated so that its moment of inertia about horizontal and vertical axes equals $1/n^4m$ times the moment of inertia of the corresponding truss of the structure. In the case of the Mount Hope bridge these moments of inertia for the structure would be for one vertical truss of two chords and for one horizontal truss of two chords. The wind-panel loads may be applied to the model by use of weighted strings operating over pulleys, as was done for the models of the Trans-Bay bridge.

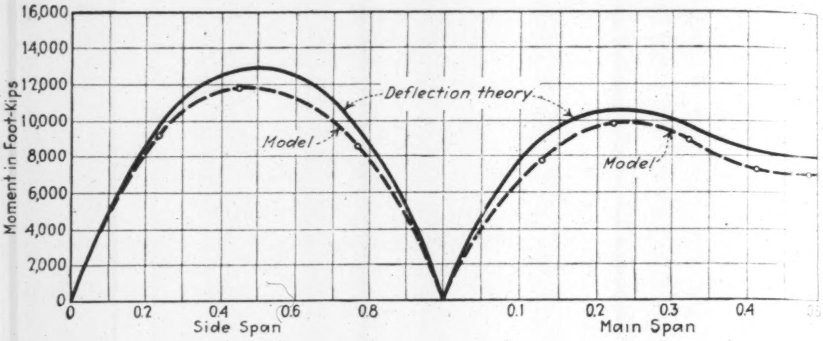

Fig. 6—Maximum positive bending moments in stiffening truss of Mount Hope bridge model were uniformly 10 per cent below values obtained by use of deflection theory calculations, because latter assume erroneously that suspender pulls are equal.

Furthermore, it appears practicable to resort to model analysis for the design of suspension bridges of novel but useful types too difficult for mathematical analysis. The theory for proportioning the sections and loads for models of more complicated structures subjected to complex systems of forces would be the same as has been employed in the case of the model described.

If it is desired to simulate stresses and strains such as those produced by earthquake vibrations, the model is proportioned as before and loaded directly at the panel points and is shaken by vibrating each foundation in a path similar to that produced by the earthquake, but with an amplitude $1/n$ times the earthquake amplitude, and a frequency the square root of n times as rapid. Under these conditions the stresses and strains in the model will equal those of the actual structure, and the deflections will be $1/n$ times as great. This procedure follows from the laws of similitude but should be verified by extensive laboratory tests before applying it in practice.

The stresses and deflections producible by load or temperature change in a suspension bridge are obtainable by direct measurement from a model; whereas to determine such values by theory necessitates a cut-and-try solution of long and involved formulas. On the other hand, the time required to get results by test of the model, once the model is constructed, is very short. In a few minutes the stress or deflection due to any specified combination of loading or temperature may be read, with little chance of error.

The structural behavior of the bridge as a whole is much clearer to one who understands present theories and who works with a model than to one who employs theory alone.

The same model that has been used in the design may be partly dismantled and used in the solution of erection problems. Simple models employing loaded wires or chains have been employed recently for predicting erection deflections of actual suspension bridges.

* * *

Changes Written Into Model Registlation Law for Engineers

Numerous changes in the model law for the registration of professional engineers and land surveyors were passed upon at a conference of representatives of several ngineering societies and state license boards held in New York on April 15. Most of the changes approved by the conference have been before its members for several months past and have in most instances been the subject of discussion at previous conferences. They all bear upon a model registration law drafted in 1930 by a committee of representatives of the American Society of Civil Engineers, American Society of Mechanical Engineers, American Institute of Electrical Engineers, American Association of Engineers, New York State Society of Professional Engineers and Land Surveyors, and the National Council of State Boards of Engineering Examiners. That law was indorsed in 1930 by the American Society of Civil Engineers, the American Association of Engineers and the American Institute of Consulting Engineers, also by several local engineering societies. The redraft, which was completed at the conference in New York, has since been submitted to the national and local engineering societies for their approval. Many of the proposed changes are in detail as to administration of the law and to clarify the wording. Important changes are made in the definitions and in the minimum qualifications for registration set up by the law. They are quite similar to those recently written into the New York State law.

Developing a Water Supply From a Polluted Chinese River

Increased per-capita consumption and restricted area for slow sand filters force company first to add coagulant to settling basins and finally to build rapid filters for million people in International Settlement, Shanghai

By H. Stringer

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EXCEPT where foreign influences dominate, most Chinese cities are still woefully deficient in sanitation. The International Settlement of Shanghai, much in the eyes of the world of late, now has a water supply far above the average of Chinese cities. This is partly due to the fact that water is provided by an English company and also to the fact that this company, having an unusually difficult matter to deal with (similar in some respects to many of those in this country), has gone far in adopting American methods of treatment. As the following article shows, plain sedimentation in fill-and-draw basins has given way to coagulation in continuous-flow reservoirs, and slow-sand filters have been replaced by rapid filters. The design of the reservoir bottoms and the methods of removing sediment, as well as the design of the filters, raise many points of general interest.
—EDITOR.

A DOZEN YEARS AGO a decided upturn in the per-capita consumption of water by the million people in the International Settlement of Shanghai demonstrated that the existing methods of plain sedimentation on the draw-and-fill system could not provide water of a quality that the slow-sand filter beds could handle at the rates the consumption required. Treatment with alum and reconstruction of the basins to care for the excessive depositions were followed ultimately by the construction of rapid filters to augment the supply. Many other extensions such as new intakes, pipe lines and additional pumps of steam (diesel and electric driven) have had to be installed, but this article will take up only the provisions for new treatment methods. The subsidence basins had to be revamped with baffles and sloping bottom hoppers, or channels, in the change from the fill-and-draw method to the speeded-up continuous operation giving heavier deposits. Due to foundation-settlement difficulties filters are grouped in small units. Washing methods utilizing the recently discovered sand-expansion factors, underdrain necessities, and operating experiences with both settling basins and filters will be described.

The International Settlement derives its water supply from the Whangpu River on which it is situated. This water has an average turbidity of 223 p.p.m. and an average bacterial content of 80,635 at 20 deg. C. and 50,072 at 37 deg. C. Shanghai, therefore, is in the unenviable position of having one of the most turbid and probably the most polluted sources of water supply in the world. The purification of such a source of supply consequently presents a many-sided problem quite apart from the abnormal growth in water demand.

Following the unusual rise in demand in 1920, coagulation was adopted, using imported aluminum sulphate with a 17 per cent aluminum oxide content. The high rate of increase in water demand persisted, and in 1924 it became apparent that increased capacity, both in settlement and filtration, was required. An extensive reconstruction program was consequently put in hand in 1925 after small-scale experiments had been made in methods of continuous settlement and rapid filtration. Water is pumped from the river to settling tanks, pumped to rapid filters and finally pumped to the town mains. The slow-sand beds are practically all fed by gravity from the settling tanks.

Continuous-settlement tanks

The success of the continuous method of operating settling basins depends on adequate mixing of the alum dose and accurate regulation of the water velocities in the tank and correct slopes to

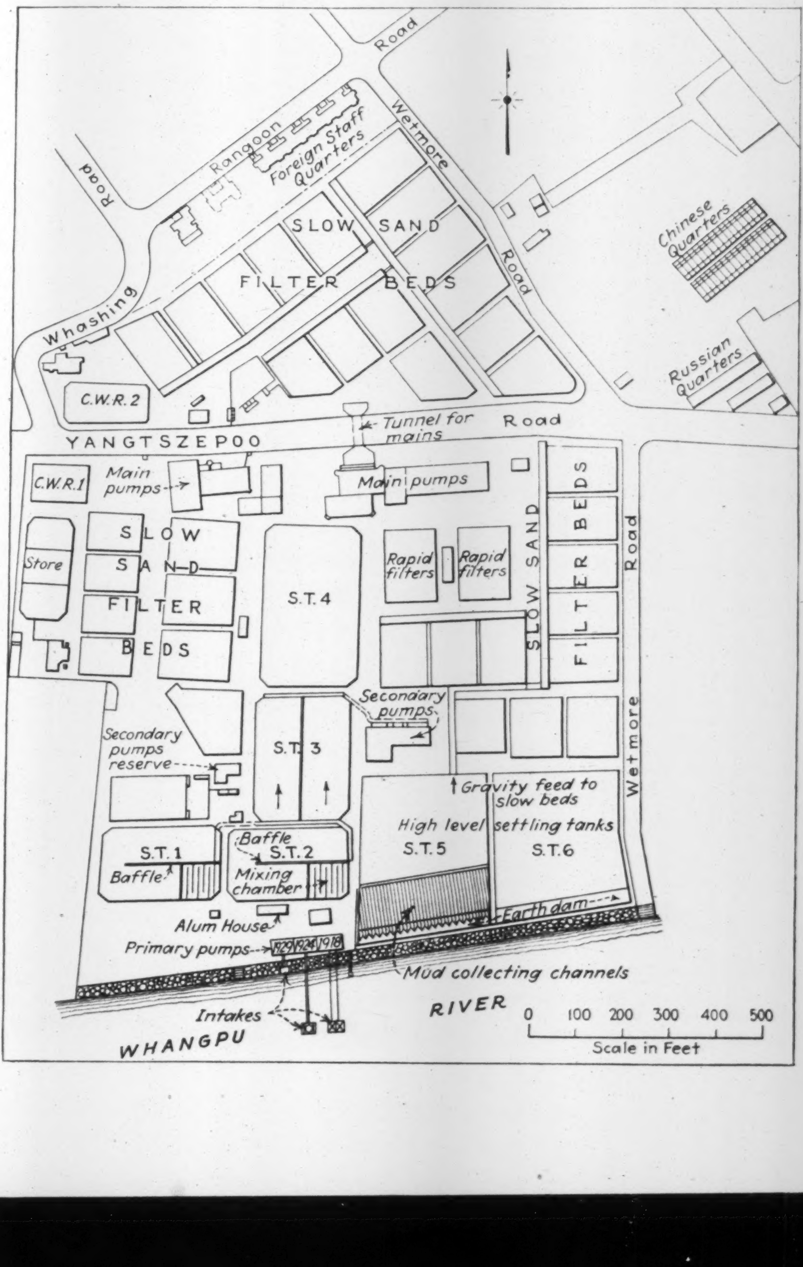

Road
Rangoon
Foreign Staff Quarters
Wetmore
Road
SLOW SAND
FILTER BEDS
Road
Chinese Quarters
Whashing
C.W.R. 2
Russian Quarters
Tunnel for mains
YANGTSZEPOO
Road
C.W.R. 1
Main pumps
Main pumps
Store
SLOW SAND FILTER BEDS
S.T. 4
Rapid filters
Rapid filters
SLOW SAND
FILTER BEDS
Road
Secondary pumps
Secondary pumps reserve
S.T. 3
Gravity feed to slow beds
Wetmore
High level settling tanks
S.T. 5
S.T. 6
S.T. 1
Baffle
S.T. 2
Baffle
Mixing chamber
Earth dam
Alum House
Primary pumps
1929
1924
1918
Mud collecting channels
Intakes
RIVER
WHANGPU
0 100 200 300 400 500
Scale in Feet

Fig. 1—Shanghai waterworks, located on the Whangpu River, were so restricted as to area that rapid filters and more efficient settling were dictated.

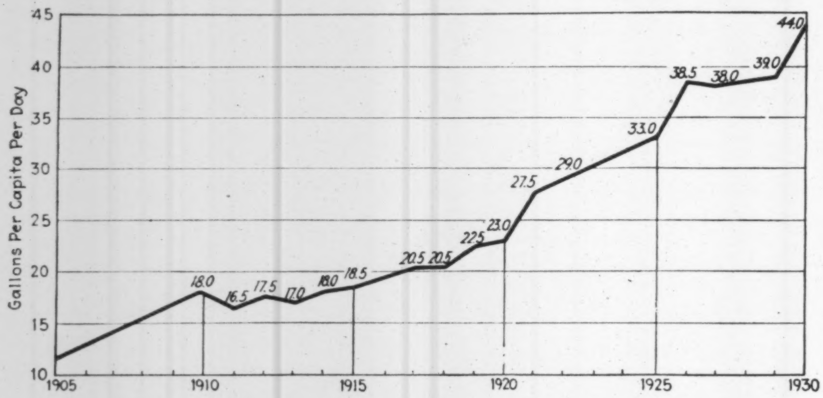
the mud pockets, which will insure flow of the deposited mud through the pipes connected to the mud pump. The velocities adopted through the tank are as follows:

Mixing chamber	— 6.36 in. per second. Mixing time of 14 min. with six channels.
Tank	— 1.00 in. per second, giving a contact period of 80 min.

To obtain this contact period, the tank is baffled by a central wall.

Factors Influencing Design—As continuous settlement would give three times as much water as that given by the old method of draw and fill, with a maximum period of six hours' quiescence, conversion of the old tanks was obviously indicated. This construction could be carried out piecemeal during the winter when demand was low, a more ambitious program being forbidden by the fact that periods of quiescence had fallen as low as half an hour during maximum demand in the summer months.

Fig. 2—Per-capita consumption of water in Shanghai for all purposes. After 1920 there is an abnormal rise in rate.



With the adoption of continuous settlement, the problem of continuous mud removal was inevitably bound up, since a deposition of mud at three times the rate under the old method of working would have tied up the tanks at too frequent intervals for cleaning by hydraulic sluicing. Every spring and autumn this method of cleaning closed down each of the tanks when worked under draw-and-fill conditions for a fortnight to three weeks, depending on the size of the tank.

The factors influencing the design of the first mud-removal system were, first, the impossiblity of gravity sluicing to the river with the existing tanks. These tanks were 16 ft. deep and, even if raised to the minimum depth of 10 ft. required for continuous settlement, certain stages of the tide would have rendered gravity sluicing inoperative. As a consequence it was necessary to adopt mud pumps. A second factor in design was the physical properties of the deposited mud. With a mud with a 40 per cent dry-solid content, the angle of repose ranged between 10 to 15 deg., while with a 75 per cent dry-mud content, angles of repose vary from 24 to 19 deg. with increasing pressure. Experiments in regard to angle of repose were carried out with the water-pressure conditions that would obtain under working conditions. As a result of these tests a slope of 20 deg. was given to the mud-collecting pockets. In practice, however, these slopes proved insufficent, and mud has to be assisted down these slopes by hand-operated shoes.

The following are the general dimensions at the settling tanks:

Tank...........	256× 150× 16 ft. deep
Main baffle wall	199 ft.
Six baffles in mixing chamber...	10-ft. centers
Mud pockets.....	36 ft. square with slopes to central valves of 20 deg. These slopes were formed of 4-to-1 lime-and mud filling, with a 4-in. slab of concrete on top

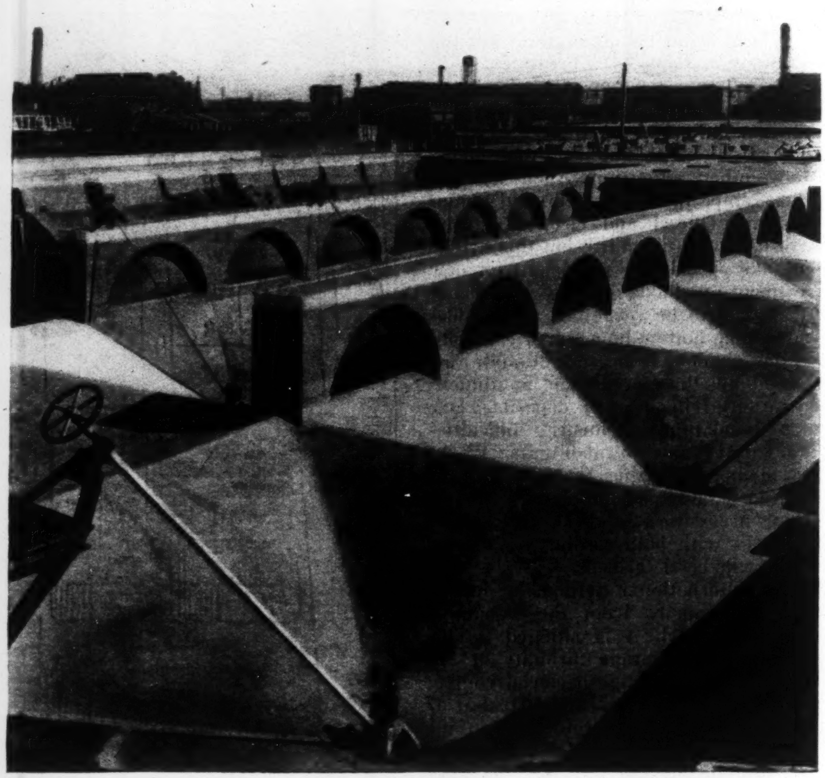
Piping—Four mud-pump collecting pipes of 9-in. diameter are connected with a 9-in. manifold housed in a 9-in.-wide culvert to allow of rodding the pipes through hatch boxes. These have never been required in practice.

The work was carried out in tank 2 in 1926, and tank 1 was converted to the same use in 1929. The conversion of this latter tank was similar in design except that mud pockets were omitted on the outlet side of the baffle wall, as it was found that 95 per cent of the mud was deposited in the first half of the tank. This effected a saving of fourteen sets of valve gears with the necessary collecting pipes. It was also found in practice that the mixing channels silted up until a channel area of about 60 sq.ft. was attained, with a velocity of 12 in. per second. The depth of the mixing channel was thus reduced to 6 ft. below working level.

These two tanks, worked at the above rates, each give 36 m.g.d. of settled water, with a turbidity of 50 at the outlet. The effluent from both these tanks is passed through a third tank, where an extra contact period is obtained. The final turbidity is by this means decreased to 30 on delivery to the filter beds. The velocity in this third tank is 0.5 in. per second, giving an additional contact period of 100 min.

Further continuous settlement capacity was also provided in 1927 as the

Fig. 3—Old low-level continuous-settling tanks were equipped with baffle wall, hopper pockets and mud valves.



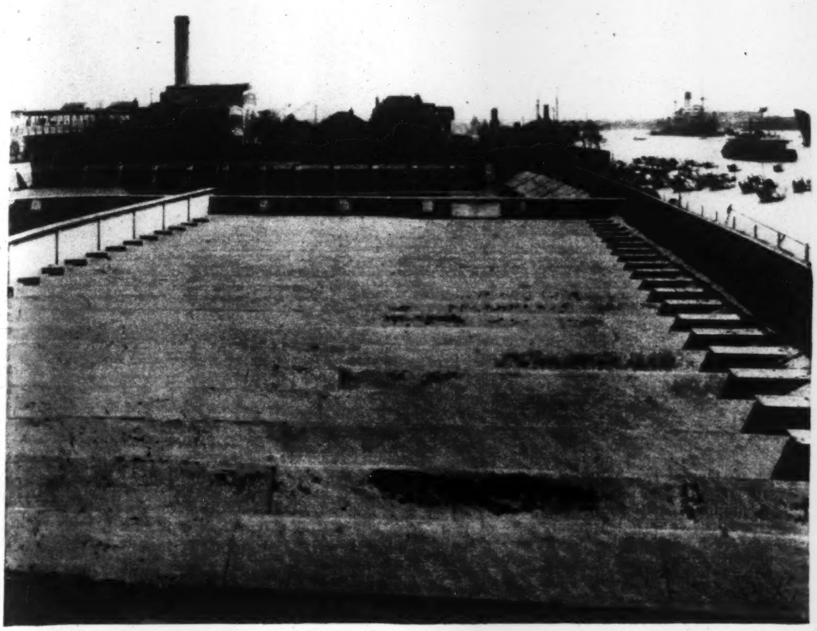


Fig. 4—Gravity mud-collecting channels discharge through dike at right directly into Whangpu River. At 3-ft. intervals 1-in. holes, in trough between ridges, lead to a discharge channel.

result of the necessity to repair the gravity dam that formed one of the walls of settling tanks 5 and 6. As these dams developed serious leakage at levels of 14 ft. and had a working level of 18 ft., it was decided to eliminate secondary pumping from the settling tanks to the filter beds by raising the dam level to admit of a working level of 22 ft., thus providing a gravity feed from the settling tanks to the slow-sand filter beds, which have a maximum demand of 36 m.g.d. The dam was light in section with slopes of 1 to 1.75 and a 10-ft. top width, but it was stable enough for the conditions required.

It was, therefore, decided to try to eliminate leakage by removing the pitched face and replacing it by a 6-in. slab of reinforced concrete. As the clay of the dam was found to be in excellent condition, this slab was designed for waterproofing purposes only, its strength in shear being only equivalent to a plaster of mud 3 ft. thick up the face of the dam. As an additional safeguard to counteract any danger of upward pressure, the toe of the dam was sheeted with 4-in. birdsmouth sheetpiling 15 ft. long. Expansion joints of sheet zinc in the 6-in. slab were placed at intervals of 50 ft.

To check leakage further and to improve conditions in the rather weaker sections at the base of this dam, there was added a plaster of mud rammed in 6-in. layers, the top level of this mud being 10 ft. above tank bottom and having a slope of 2 to 1. Since this work was done, leakage has decreased progressively and has moved downward. It in now almost negligible. The dam has been in continuous use since the spring of 1929 and has resulted in an annual saving in secondary pumping of $6,455.

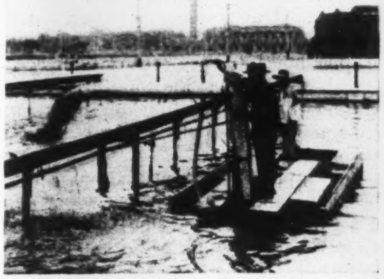


Fig. 5—Mud channels also are cleaned by air lift, discharging into temporary wooden channels supported on the ridges of the mud channels.

In other essentials the principles of design are the same in this tank as in tanks 1 and 2, but in this case a different method of mud removal was adopted owing to the higher working level of the tanks and the possibility of frequent gravity sluicing into the river. The suspended matter here settled into a series of 23 channels, each 103 ft. long and placed at right angles to the direction of flow. These channels, at 12-ft. centers, are triangular in section, 5½ ft. deep with 45-deg. slopes. The bottom width of the channel is 9 in. At 3-ft. intervals the bottom is pierced with 1-in.-diameter holes, through which the mud flows to a 6x6-in. collecting channel. These channels have a fall of 6 in. and are closed by 6-in. valves at their junction with a concrete manifold 1 ft. wide by 1 ft. deep, with a fall of 1 in 75. This manifold conveys the mud to two 20-in. pipes, which go through the dam to the river 6 ft. above the tank bottom. In order to keep the mud in motion when sluicing, each channel is provided with a set of hand-worked scrapers. This method of mud removal, which was adopted as a result of lengthy experiments, has not as yet proved an entire success, and an air-lift pump fixed to a float has been used to supplement the gravity sluicing. This pump delivers into triangular wooden channels fixed to the ridges of the existing mud-removal channels. The total settlement capacity is now 96 m.g.d., with a future capacity of 156 m.g.d. that will be possible without further alterations.

The estimated cost per million gallons of removing mud by the discontinuous process was, in 1927, 16.9c. for operation and 11c. for capital cost, while by the continuous method the total cost is 31.4c. per million gallons, of which 6.9c. is for operation and 24.5c. for capital. Mud disposal also involves the company in additional expense, as payment has to be made for dredging the material pumped back into the river. This added 17.3c. per million gallons to settlement costs in 1929, at about 41.2 per cent load factor.

Filtration

Apart from other considerations, the abnormal increase in land values and the dilatory methods of conducting land deals in the Chinese manner practically compelled the adoption of methods of rapid filtration. In this process the usual rate of working filters is 100 gal. per square foot per hour, as against a maximum of 5 and an average of about 2 in slow sand beds. Thus, though the capital cost in 1923 of slow sand filters was only 77c. per square foot, as against $11.08 for the rapid filters constructed in 1925, the capital cost for equal capacity of slow sand beds would have been at least $41.08—quite apart from cost of land if it could have been obtained in time.

The operation cost of rapid sand filters is, however, higher than that for slow sand beds—$1.07 per million gallons as against $1.49 in 1929. The average demand in 1929 on the rapid filters was 411.6 m.g.—that is, 57.2 per cent load factor, giving an extra operating cost of about $2,000 per annum, which adds, at 7 per cent, a further capital expenditure of $30,000 to rapid filtration, thus bringing the cost per square foot up to $13.80.

Rapid Filters—Two batteries of eight rapid filters have been constructed since 1925, both of the same capacity—24 m.g.d. Generally the designs are similar, but in the battery constructed in 1925 experience showed that our expectation of algæ growth on these filters would not be realized and that these need not be covered. This alteration admitted of raising the reservoir capacity beneath the filters to 750,000 gal., as against 500,000 gal. in the first battery, giving a longer contact period for chlorination.

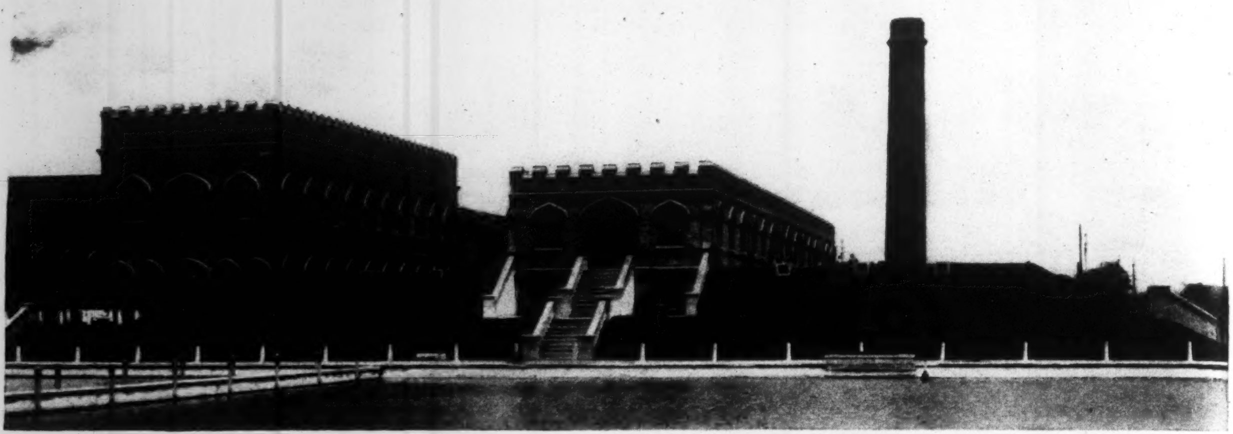


Fig. 6—One of two batteries of rapid filters with rectangular washwater tank at left. To minimize settlement, eight filters are designed as four independent units of two filters each, with independent pipe gallery and operating floors between each set of four filters.

with consequent improvement in the bacterial quality of the effluent. Further, some simplification in piping and saving in valves resulted in the adoption of a two-way valve instead of individual valves for influent, sewer, washwater and effluent pipes. Again, as a result of our experience of foundation settlement in the first battery, the eight new filters are designed as four independent units of two filters each, while the pipe gallery and operating floors between each set of four filters are entirely independent units. A further improvement was the substitution of steel and corrugated iron for concrete in the roof of the operating floor. Lastly, in order to admit of isolation of each set of rapid filters, the effluent channels were built in duplicate.

The whole building in plan consists of three independent units constructed on separate foundations. All filters are 40x41½ ft., c. to c. of walls, by 8 ft. 3 in. high, not covered on top. As the efficiency of operation depends almost entirely on the arrangements for backwashing, the following details as to the combined underdrain and backwash system are given.

Each filter has an area of 1,400 sq.ft. and is divided into two halves and fed by a central duct, 3 ft. wide with a 3½x2½-ft. clear-water duct underneath. The underdrain and backwash system is designed to what is known as the Illinois standard.

There are seven washwater troughs per filter fixed at 5 ft. 10 in.-centers and 6½ ft. above filter-floor level, with a freeboard of 2 ft. from sand level to lip. The troughs are constructed of reinforced concrete, 2 in. in thickness, with a semicircular bottom of 7-in. radius inside with a rectangular section 12 in. deep by 14 in. wide above. They are constructed with a fall of 3 in. in their length of 17 ft. 2 in. to the central duct. The sizes of these troughs are based on an expansion of 45 per cent with the sand selected. The underdrain and washing system consists of 48 laterals per half filter fixed 10 in. apart. These are 4-in. cast-iron pipes, with 35 holes per lateral (7/16-in. diameter) at 6-in. centers. Experiments with an underdrain system in porous concrete proved a failure in practice. The system was designed on the following basis: total area of the strainer holes, 0.2 per cent of filter area; laterals three times the total area of the strainer holes, manifold or duct, one and one-half times the total area of the strainer holes.

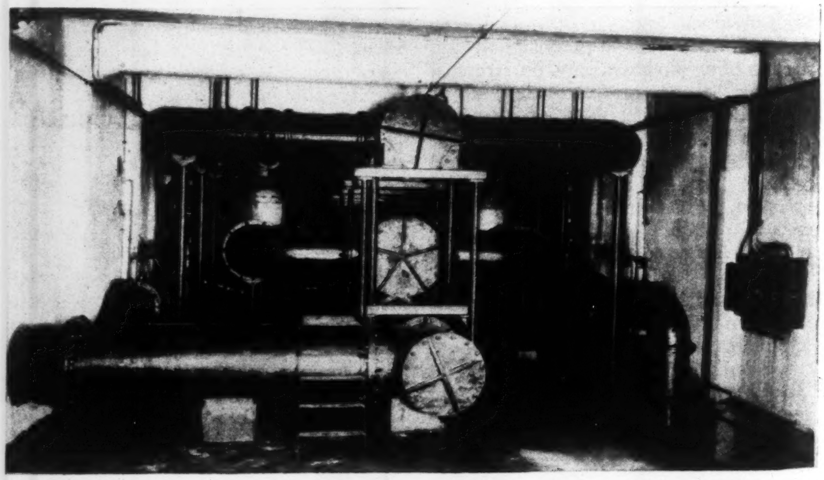
For rapid-filter sand the sand from the slow sand filters yields 44 per cent of a sand within the required limits, its effective size being 0.44 m.m. and its uniformity coefficient 1.82 A 30-in. depth of sand was placed over a five-layer 18-in. gravel underdrain.

In operation these filters have been uniformly satisfactory. The rate of working is 100 gal. per square foot per hour, and filters are washed at a 6-ft. loss of head, but always once in 24 hours, the percentage of washwater used averaging 3.62 per cent in 1930. The bacterial reduction from river to filter effluent is 99.51 per cent, and after chlorination at the filters it is 99.99 per cent. Prechlorination was introduced in the summer of 1929, and this has reduced the bacterial load on the filters by 90 per cent, with consequent further improvement in the effluent. The average dose used is 0.5 p.p.m. Chlorination is also used on all suctions of the main pumps, the average dose being 0.2 p.p.m.

Capital expenditures in the interval 1926-1930 were $1,244,813 for a rise in maximum hourly demand of 24 m.g.d., implying an annual capital expenditure of $248,963 to keep pace with increase in water demand.

Distribution—The mileage of mains at the end of 1930 was 171.01. The main-laying program for each year is fixed by probable maximum demand, checked by pitometer surveys made every two years. As regards pipes, all larger mains are now of steel, unlined but protected outside by wrapping, a minimum cover of 3 ft. being provided everywhere. In common with other waterworks undertakings, increased vibration due to heavy motor traffic is

Fig. 7—Rapid-filter pipe gallery of the Shanghai waterworks. Raw-water mains are at top, drain is in center, and washwater and clear-water pipes and water controllers are near floor.



having its usual effects. Thus leaks per mile of main increased from 3.20 in 1929 to 4.29 in 1930. At present all industrial supply is metered. Local conditions demand a wet-dial meter where possible in the smaller sizes, but above the 3-in. size a dry-dial meter is necessary.

All the works referred to have been carried out to the designs and under the direction of C. D. Pearson, engineer-in-chief and manager of The Shanghai Waterworks Co. Ltd., to whom the author is indebted for permission to make use of the above information.

Letters to the Editor

Public Works or Relief?

Sir—In your last issue there was presented a public-works program sponsored by the American Society of Civil Engineers for the creation of employment and consequent stimulation of business. Despite its promise of relief, this program leaves, in the minds of many, several questions as to its desirability and efficacy.

Unemployment relief, or to put it more baldly, starvation, destitution and want are problems of today, demanding immediately food, shelter and employment. To find this immediate answer in public works, there would have to be given to city, county, state and even national officials dictatorial powers to make commitments for public loans and to call in architects, engineers, contractors and realtors; and assign, over the table, various groups of these for the immediate commencement of work. While it is claimed that plans for many of these projects are available, a large number would certainly require additional engineering services for the preparation of details, etc. Power practically to seize property, perhaps in some extra-legal manner, will have to be delegated to real estate and legal experts. Contractors and builders will have to be selected arbitrarily. It is doubtful if even the most enthusiastic advocates of this plan can envision any such delegation of dictatorial powers under the present set-up of our government institutions.

If it is intended to go through the usual procedure of having voters approve public works, of issuing bonds, of acquiring property by the usual legal methods, of involving the letting of contracts, and the routine now prevailing, a public-works program holds out really little hope of effecting its purpose promptly for the millions of men who are now walking the streets of our cities and the byways of our country.

It is generally considered more desirable to create employment through normal commercial activity than through public works. A considerable smaller sum of public money used to revive or sustain commercial and private construction and activity would provide employment for a larger number of men with a better chance of such employment continuing or even increasing. Such use of public monies is illustrated by the recent loan through the Reconstruction Finance Corp. of $27,000,000 to the Pennsylvania Railroad for electrification work.

If public works are such a panacea for unemployment, why have they been so ineffectual in Germany and England? It is alleged that of the hundreds of millions borrowed from this country by Germany a large part of every dollar has been invested in a most extravagant program of public works, slum removal, waterworks, etc. Yet, according to figures recently published, unemployment in Germany today has nearly doubled in the past decade. Similarly, England several years ago embarked upon a widespread system of housing construction looking to the relief of its unemployed; but finally turned to direct relief in the form of the much-criticized dole.

These failures in themselves are certainly a warning to be sure that conditions in this country are sufficiently different politically and psychologically to insure that a public-works program would really accomplish the purpose of relieving unemployment promptly on a large scale.

Passing to the effects of this plan on general business, to accomplish its purpose such a program must be put into effect immediately and fully. As its foundation, however disguised, is a large increase in public debt, this would mean the immediate flooding of the investment market with new bond issues to the huge total of a billion and a half dollars in local or possibly national issues. Such bonds, to be attractive, would presumably have to bear a higher rate of interest than similar current obligations and even then possibly require an emotional appeal to patriotism comparable to the sale of Liberty Loans. This would cause a further unsettlement in existing values of public and private investments issues.

From the very nature of these enterprises, they must be created in their entirety with full-fledged overhead charges of interest and amortization, starting practically from their commencement. In other words, the interest and amortization charges of many of these works would have to be added, for a time at least, to the tax burden of the people, at a time when nearly everyone is hard put to meet current expenses.

While intelligent men are emphatically convinced of our government's financial soundness, many sinister influences are seizing on every pretext to undermine public confidence in the United States gold and financial positions. A huge program of forced borrowing would add fuel to this fire.

There is one further phase that may well be considered—our moral responsibility to the next generation and possibly even their children. While we are unquestionably, in some measure, paying the price ourselves of the economic folly of the World War, we, and all the other peoples of the world, are passing on a terrible burden of debt and obligation. Because of our own present distress, have we the right further to borrow ourselves out of difficulties and pass an added problem on for solution? It may be answered that the debts now proposed will be self-sustaining, but, as before stated, they are a forced charge on the income of the country, and in so far as they are created in advance of normal requirements and needs they would constitute a serious and abnormal demand upon the country's income.

It is fully recognized that many constructive and intelligent influences may be properly used to quicken the action of economic re-adjustment. Such efforts, however, should be more in the nature of aiding, sustaining and stimulating normal commercial and business activity and values. To this end, the Reconstruction Finance Corp., with its constantly widened powers to help private business, is now effectively working. Will a huge public-works program at this time help or hinder these constructive measures already set up?

In conclusion, the writer wishes to state that he fully appreciates the importance of public works in the recovery program of the country provided they are not of the hot-house variety—that is, forced or stimulated as additional facilities upon communities not actually in need of them. Also, that where *needed public works* may be *promptly* undertaken by communities that are able and willing to finance them, they will ultimately constitute a helpful factor for unemployment relief and business activity. But this is entirely different from the national program of public works, induced upon communities by the national government and backed by the implied use of national credit on a huge scale.

Furthermore, in the matter of unemployment and destitution, why not face the real fact that direct relief through public monies to supplement private funds will have to be provided immediately?

The writer regrets that the limitation of space prevents a discussion of the existing agencies and forces making for business recovery and a consideration of certain further measures that would contribute, through public monies and legislation, to the further revival of business and commercial activities.

WM. JOSHUA BARNEY.

New York City,
June 6, 1932.

NEWS OF THE WEEK

Accident in River Cofferdam Declared Unavoidable

In the opinion of the district engineer in immediate charge of the work on the government dam at Marmet Lock, on the Great Kanawha River, the recent land slide there which caused the death of six men (*ENR* Apr. 28, p. 631) resulted from "an inherent hazard in this class of construction work." The contractor, the district engineer, says had taken every reasonable precaution to properly slope the bank and protect his men. Major Fred W. Herman, the district engineer who made the report, holds positively that the accident was unavoidable.

The Dravo Contracting Company of Pittsburgh, the contractor on this project, is engaged in the construction of lock B at the site mentioned. Foundations were being prepared for the placing of concrete. The common excavation had been removed from the rock surface to permit of the channeling and drilling of the rock. The slide was caused by ground water softening a clay pocket in the bank. There had been some slides at this point during the high water period in March but the material in those slides had been removed and new bank slopes had been established.

There were two slides on April 23, the date of the accident. The first occurred at 9:30 p.m. when 900 cu.yd. of earth slid into the excavated area. It covered two men who were operating a jack hammer and a channeling machine. At 9:55 p.m. when 25 men were in the hole digging for the covered bodies a second slide of 1,100 cu.yd. of earth, occurred. The two men first covered died as did four members of the rescue squad. Seven other members of the squad were injured. Between the time of the first and second slides an inspection had been made of the bank and there was no evidence of a further movement of material.

◆

Winter Road Maintenance Found to Pay in Canadian Provinces

The annual interprovincial highways conference sponsored by the Canadian Good Roads Association was held in Ottawa, May 30 and 31. In addition to the delegates from the federal and provincial governments, representatives were present from the Board of Railway Commissioners, the Canadian Pacific Ry., the Canadian National Ry., the Canadian Engineering Standards Association and the Canadian Automobile Association.

Discussion on winter road maintenance brought out the statement that there is a constantly increasing demand on the part of the public for winter roads. The Quebec Roads Department maintained 387 miles of roads last winter at a cost of $250 per mile, while Ontario maintained 3,000 miles at a cost of $75 per mile. Ontario's experience was that the winter tourist traffic was increasing yearly, and that last winter its volume was 40 to 50 per cent of the summer traffic. The Ontario system of winter maintenance is self-supporting, the cost being fully met by the increased collection of gasoline tax.

A report was forwarded from Nova Scotia on a series of tests of dust layers or gravel road preservatives conducted in that province. The three materials used were oil, calcium chloride, and Malagash salt, and of these the calcium chloride was stated to have proved most satisfactory. This experience coincided with that of other road authorities.

Reports presented showed that legislation affecting highways and public safety passed in various provinces during the past year included reciprocity in commercial registrations, increasing headlight beam lengths to 200 ft. from 150 ft., financial responsibility measures and regulation of tourist camps and information bureaus.

◆

Chinese Dykes Reconstructed

Approximately 2,000 miles of dykes have been reconstructed in the Chinese flooded areas in recent months according to a report to the Department of Commerce. The reconstructed dykes are expected to provide adequate protection unless the summer has some abnormally heavy rains. The Commerce Department is also advised that normal railway passenger service has been resumed in the Shanghai area.

New Port for Ocean Vessels at Albany, N. Y., Dedicated

New facilities for ocean-going vessels which have been under construction at Albany, N. Y., during the past six years, were formally dedicated June 6-7. Opening of the port to ocean-going vessels has been made possible by the deepening of the river carried on by the federal government and completed late last year. The river is now open to vessels of 27 ft. draft from New York to Albany, a distance of 143 miles.

The new port at Albany has a wharfage of 4,400 ft., a harbor depth of 35 ft., a turning space for 1,000 ft. vessels and 250,000 sq.ft. of transit sheds and warehouses. The principal feature of the port is a grain elevator of 13,000,000 bushel capacity, said to be the largest single-unit elevator in the world. Opposite Albany, on the Rensselaer side of the river, is a modern lumber terminal with storage capacity for 100,000,000 f.b.m. of lumber.

Part of the port has been in use for some time. It is expected that it will develop a large business in handling freight coming down from the Great Lakes by both rail and barge for transshipment to ocean-going vessels. The total cost of the project is placed at $18,500,000.

Arnold G. Chapman, Albany, is engineer for the Port District Commission

◆

Colorado River Aqueduct Bonds Approved by State Supreme Court

THE California Supreme Court, in a decision rendered June 2, upheld the validity of the $220,000,000 bonds to be issued by the Metropolitan Water District of Southern California to finance the construction of an aqueduct to bring water from the Colorado River to cities of southern California. The decision becomes effective 30 days after its publication.

In September of last year, the 13 cities composing the Metropolitan Water District of Southern California, by a vote of 5 to 1, authorized this bond issue to build the aqueduct. In November a taxpayers' suit was begun in Los Angeles and subsequently on appeal from the favorable decision of the lower court was taken to the State Supreme Court in order to establish the validity of the bonds before undertaking to market them.

The project calls for an aqueduct designed for an average flow of 1,500 sec.-ft. from the Colorado River near Parker, Ariz., over the high areas lying between the river and the Los Angeles basin, a distance of approximately 240 miles. Power for pumping will be obtained principally from Hoover Dam. The aqueduct will be made up of tunnels, open channels and pressure conduits.

F. E. Weymouth is general manager and chief engineer of the Metropolitan Water District of Southern California and at the present time Julian Hinds, the chief designing engineer, is acting chief engineer.

W. P. Whitsett, chairman of the board of directors of the district, in commenting upon the court decision, said: "Until this decision has been made final, it will be impossible, of course, for the district to proceed with the sale of any of the aqueduct bonds. Work on the aqueduct cannot be started until a block of these bonds has been sold. In the meantime, every effort is being made by officers of the district to hasten any step that needs to be taken to set the work actually under way.

"At the present time, the Metropolitan Water District is presenting to Congress and federal officials the urgent recommendation that the aqueduct bond issue, in whole or in part, be taken up by the Reconstruction Finance Corporation in Washington, and the financing of our water project be thus hastened. Because of the fact that the Colorado River aqueduct will call for the purchasing of approximately $100,000,000 in materials, supplies and equipment, and will provide another $100,000,000 in payrolls for about 10,000 men over a six-year period, we feel that this undertaking is one which most properly should be encouraged and facilitated through the medium of the federal financing corporation."

Supreme Court Makes Significant Ruling in Kansas Motor Vehicle Case

THE MOTOR vehicle law of Kansas, passed in 1931, was upheld by the United States Supreme Court on May 23 —as briefly noted in *Engineering News-Record* of May 26. Because the Kansas law represents one of the first attempts by a state to regulate the operation of private motor vehicles on its highways, the Supreme Court findings are of nation-wide significance. The opinion of the court was rendered in an appeal from a final decree of the District Court of the District of Kansas which dismissed the bill of complaint in a suit brought to restrain the enforcement of the motor vehicle act of Kansas by the Continental Baking Co. and others who are described as "private motor carriers of property," operating bakeries in Kansas and other states and making deliveries to their customers by their own trucks.

The statute in question relates to motor vehicles when used upon any public highway of the state for the purpose of transporting persons or property. It applies to those who are engaged in such transportation as "public motor carriers" of property and passengers, "contract motor carriers" of property and passengers, and "private motor carriers of property." A public motor carrier is defined as one transporting for hire as a common carrier having fixed termini or routes, a contract motor carrier of property is defined as one who is not a public motor carrier and is engaged in transportation for hire as a business, and a private motor carrier of property is one transporting "property sold or to be sold by him in furtherance of any private commercial enterprise."

"The act does not apply to (1) motor carriers operating only within any city or village of the state, (2) private motor carriers operating within a radius of 25 miles beyond the corporate limits of the city or village, (3) the transportation of livestock and farm products to market by the owner thereof or supplies for his own use in his own motor vehicle, and (4) the transportation of children to and from school.

Public motor carriers in intrastate commerce must obtain certificates of convenience and necessity. Contract motor carriers and private motor carriers of property either in intrastate commerce or in interstate commerce must obtain licenses. Applications therefor must give information as to ownership, financial condition and equipment, and such further facts as the Public Service Commission may request. The commission is required, upon receipt of this information and on compliance with the regulations and payment of fees, to issue a license.

In addition to license fees, public motor-carriers, contract motor carriers, and private motor carriers of property must pay a tax of five-tenths mill per gross ton mile. Every motor carrier covered by the act must keep a record, upon prescribed forms, of all vehicles used and must certify under oath summaries showing the ton-miles traveled monthly and such other information as conditions may require.

Of the money received under the provisions of the act, 20 per cent is to be applied to administration and enforcement and the remainder is to be placed to the credit of the state highway department. No certificate or license is to be issued by the commission to any of the described motor carriers until a liability insurance policy approved by the commission has been filed.

The general situation to which the statute is addressed is thus described by the district court: "The state of Kansas has constructed at great expense a system of improved highways. These have been built in part by special benefit districts and in part by a tax on gasoline sold in the state and by license fees exacted of all resident owners of automobiles. The public highways have become the roadbeds of great transportation companies, which are actually and seriously competing with railroads which provide their own roadbeds; they are being used by concerns such as the plaintiff for the daily delivery of their products to every hamlet and village in the state.

NEW YORK'S NEWEST SKYSCRAPER

By virtue of a taller flagpole the new 67-story Cities Service Building in the New York financial district replaces the nearby Manhattan Company building (in the background of the picture) as the third tallest building in the world, exceeded only by the Empire State and Chrysler buildings. The outstanding innovation in the Cities Service Building is the use of 8 double-deck elevators developed by the Otis Elevator Co. and said to increase the rentable area by 48,000 sq.ft. over a single-car installation. The lower deck of a car will serve odd numbered floors, the upper deck the even numbered floors. The building also includes ten escalators, serving the lower floors and a unit ventilating system drawing outside air over radiators beneath the windows and delivering it to all rooms in wall and ceiling ducts.

"The highways are being pounded to pieces by these great trucks which, combining weight with speed, are making the problem of maintenance well-nigh impossible. The legislature but voiced the sentiments of the entire state in deciding that those who daily use the highway for commercial purposes should pay an additional tax. Moreover, these powerful and speedy trucks are the menace of the highway."

It is apparent said the Supreme Court that Kansas in framing its legislation to meet these conditions did not attempt to compel private carriers to become public carriers. The legislation did not purport to put both classes of carriers upon an identical footing and subject them to the same obligations. It recognized and applied distinctions. Public or common carriers and not private carriers are required to obtain certificates of 'public convenience and necessity.' The former and not the latter are put under regulations as to fares and charges. It does not appear from the bill of complaint that any regulation had been prescribed, or that the commission has made any order, of which private carriers may complain.

"The statute itself, however, does impose certain obligations upon private motor carriers of property, and the first question is whether these provisions violate the constitutional restrictions invoked.

"Private motor carriers of property must obtain a license, pay a tax, and file a liability insurance policy. The Public Service Commission has no authority to refuse a license if the prescribed information is given and the liability insurance policy filed. It is not shown that either regulation or license fee are unreasonable. The tax and license fee, over the expenses of administration, go to the highway fund of the state for the maintenance and reconstruction of the highway the carrier is licensed to use.

"The insurance policy is to protect the interests of the public but only to protect third persons from injuries to their persons or property.

"Requirements of this sort are clearly within the authority of the state which may demand compensation for the special facilities it has provided and regulate the use of its highways to promote public safety. Reasonable regulations to that end are valid as to intrastate traffic and, where there is no discrimination against the interstate commerce which may be affected, do not impose an unconstitutional burden upon that commerce.

"Whatever uncertainty may exist with respect to possible regulations of the commission will be resolved as regulations are promulgated. If any of these transcend constitutional limits, appellants will have their appropriate remedy. The provision as to keeping records and to maintaining uniform methods of accounting may in the case of private carriers be assumed, until the contrary appears, to have relation, as the state authorities assert, to determination of the amount of the tax to which the private carriers are properly liable."

Bids Made for Flood Control Bonds

An offer to purchase $2,000,000 in bonds issued by the county board of supervisors, Los Angeles, Calif., for the initial work on dam No. 1 of the county flood control project was made by the Bank of America June 3. No bids were received when the bonds were first offered on June 1.

The estimated cost of constructing dam No. 1 is $10,000,000. Exploration work at the site of the dam has been in progress for some time. Contract for the construction of dam No. 2 was let in March.

The original flood control project on the San Gabriel River called for a dam 500 ft. high at the so-called Forks site. After considerable work had been done upon the preparation of that site it was abandoned on account of unfavorable geological formations and the construction of two smaller dams at more favorable sites was determined upon.

◆

Unusually Low Typhoid Record in American Cities in 1931

Twelve of the 93 American cities of 100,000 population and over recorded no typhoid deaths in 1931, according to the twentieth annual report on "Typhoid in Large Cities of the United States" published by the *Journal of the American Medical Association* on April 30. Arranged geographically these cities were: Cambridge, Fall River, Lynn and Somerville, Mass.; Waterbury, Conn.; Utica, N. Y.; Reading, Pa.; South Bend, Ind.; Flint, Mich.; Wichita, Kan.; Des Moines, Iowa; and Long Beach, Calif. Fifty of the remaining cities had death rates of from 0.2 to 1.9, making 62 of the 93 cities, or exactly two-thirds, in the 0 to 1.9 class. Next come 23 cities with rates of 2 to 4.9; then 6 rating at 5.4 to 7.3; and finally two above 10—Atlanta, 12.6 and New Orleans, 13.9. The showing for the maximum rate cities is made better by the fact that one-third or more of their deaths were stated to be non-residents, a condition applying to 18 other cities. In five other cities all the deaths reported were classed as non-residents.

◆

Economics of Railway Track Ties Discussed by Tie Producers

Technical, economic and trade subjects were included at the annual convention of the National Association of Railroad Tie Producers, at Memphis, Tenn. An economic comparison of large and small ties was presented by E. A. Hadley, chief engineer of the Missouri Pacific R.R., in the discussion of which it was shown that as different sizes are necessarily produced, restriction of purchase to large sizes exclusively would result in material increase in prices. A comparison of different kinds of timber for ties must be based upon strength, durability and such physical charactertistics as hardness, resistance to abrasion and tendency to checking, according to a paper by Hermann Von Schrenk, consulting engineer. Protection against abrasion will extend the use of softwood ties and increase the life of treated ties. That railroads are depleting their stocks materially was shown by E. T. Howson; while they used ties valued at $70,000,000 during 1931, they spent only $47,500,000 for new ties.

The relations of tie contractors to railroad purchasing departments were discussed by R. C. Hobbs. An important development was in initiating action for compiling periodical records of ties on hand in the stocks of producers and railroads. Officers elected for 1932-1933 include: President, S. S. Watkins, of the Joyce-Watkins Co., Chicago; vice-presidents, B. N. Johnson, Richmond, Ind., and D. C. Jones, Chicago; secretary, re-elected, Roy M. Edmonds, St. Louis, Mo. It is proposed to shorten the name to Railway Tie Association, as a matter of convenience.

◆

Highway Engineers to Rotate

A system of rotating the various division engineers is contemplated by G. H. Cutler, chief engineer of the Missouri Highway Commission. If the plan meets with the commission's approval, the first general change will be made within the next few months. In this way it is expected to make the district engineers familiar with conditions to be met throughout the entire state.

◆

Finding of Original Cost by Power Commission Approved

The right of the Federal Power Commission to determine the original cost of a power development built under a license from the commission and to use its findings in place of statements of original cost prepared by the licensee was affirmed by the Court of Appeals of the District of Columbia on May 31.

The subject of the commission's rights in this matter was brought before the court by the Clarion River Power Co. which in 1922 received a 50-year license from the commission to develop what is known as the Piney site on the Clarion River in Pennsylvania. In 1930 the company filed a statement with the commission representing the cost of constructing the project at $11,032,816. The accounting division of the commission's staff investigated this statement and recommended to the commission that $6,213,904 be eliminated from this cost figure. The company protested and carried the case to court to determine whether the commission was authorized by the Federal Power Act to make such a finding. The company contended that the commission was required to accept the company's own statement of its expenditures. On this point the court said, "The statute requires the licensee to submit to the commission statements covering net investment and costs of the project, and we think it plain that the purpose of requiring the licensee to make provision for currently determining the cost is to enable the commission as the administrative body, not the licensee, currently to make such determination. It follows, therefore, that the commission in making the investigation to determine the original cost and net investment of the Clarion Company in the Piney project was acting within its jurisdiction and, hence, is not subject to the control of the court."

The point at issue has been the subject of controversy between the commission and some of the licensees for some time as the commission's findings have frequently been lower than those of the power companies.

Public Works Construction Advocated Before Chicago Engineers

The Chicago Engineers Club, May 24, at its regular Tuesday luncheon listened to Frederic Bass, professor of municipal and sanitary engineering, University of Minnesota, speak on current problems. Professor Bass advocated a plan of public work wherein enough men shall be given enough work to reverse the falling tide of business. He advised that the federal government should focus the potential power of idle people and idle money in a plan of concerted action. A plan for municipal public work of $3,000,000,000 would give work to 1,500,000 directly and 2,000,000 additional indirectly. Financing, he felt, should be federally initiated with a credit agency taking municipal bonds for self-sustaining programs only. The plan he said was not enough. There must be a belief in its ultimate success. With this determination the nation can relieve its losses, meet its obligations and establish its future, in security.

◆

Two Engineers Retire From Pennsylvania Railroad Service

W. C. Cushing, engineer of standards, and William B. McCaleb, general superintendent of water service, Pennsylvania Railroad, were retired on May 31. Mr. Cushing had been in the service of the railroad for 44 years and Mr. McCaleb for 52 years. Both entered the service in junior positions in the engineering department. A considerable part of Mr. McCaleb's time was spent in the operating department, including 15 years as superintendent at Harrisburg. Mr. Cushing is best known for his active work in the American Railway Engineering Association, of which he is a past president and an honorary member. With the exception of a few years in the operating department, Mr. Cushing's service with the railroad has been continuously in the engineering department. In 1903 he was made chief engineer of maintenance of way on lines west and in March, 1920, he was appointed engineer of standards on the staff of the chief engineer at Philadelphia.

◆

Locomotives Delivered for Newly Electrified Line

The first experimental units of the Pennsylvania Railroad's new electric freight locomotives which will go into operation upon the completion of the electrification of the railroad between New York and Washington have been delivered.

Except for the experimental steam locomotive built by the Timken Roller Bearing Co., these are the first locomotives to be built with roller bearings. The new locomotives weigh 185 tons and are designed to haul longer trains at higher speeds than has been found economical with steam locomotives. Each unit has four pairs of 62-in. driving wheels, each equipped with a motor. They are so designed that two or three units can be combined under the control of one engineman. All parts as far as possible are made interchangeable with the railroad's new electric passenger locomotives.

Dean Milo S. Ketchum Honored

The staffs of the College of Engineering and the Engineering Experiment Station of the University of Illinois gave a dinner on May 25, 1932, in celebration of a decade of progress of the College of Engineering under the guidance of Milo S. Ketchum as Dean of the College of Engineering and Director of the Engineering Experiment Station. Short addresses were made by A. N. Talbot, Assistant Dean H. H. Jordan, and Professor W. C. Huntington. Professor A. C. Willard presented an illuminated parchment to Dean Ketchum on behalf of the members of the staffs of the College of Engineering and the Engineering Experiment Station.

◆

New Road to Be Built in San Gabriel Canyon

Bids will be opened in San Francisco, Calif., on June 16 for the construction of the proposed new San Gabriel Canyon highway made necessary by the construction of Pine Canyon dam by the city of Pasadena, Calif. The highway will replace the present road along the floor of the canyon. It will be 7.1 miles long, with a width of 28 ft. The estimated cost is $1,300,000. The road is expected to be a scenic drive and will include a number of reinforced concrete bridges spanning side canyons and ravines. The city of Pasadena and the Los Angeles County Flood Control District each will contribute forty per cent of the cost. The California Highway Commission and United States Bureau of Public Roads each will pay ten per cent. The new road will be maintained by Los Angeles county.

◆

Century of Progress Exposition Dedicates Hall of Science

One year in advance of the opening of the Century of Progress Exposition at Chicago, June 1, the Hall of Science, having 7 acres of exhibit space, was dedicated by Frank B. Jewett, chairman of the science advisory committee of the National Research Council. Mr. Jewett headed a board of 400 advisory experts who outlined the scope of the particular sciences they represented. Many prominent engineers are on the list. Mr. Jewett's remarks unfolded the basic principles back of the exposition.

In addition to the Hall of Science, already standing or virtually completed of the World's Fair structures, are the Administration building, the Travel and Transport building, the replica of Old Fort Dearborn, the Radio, Communications and Electrical buildings. Construction has recently started on the General Exhibits group and the Agricultural group. A bridge over the lagoon connecting the Hall of Science and Electrical group is under way. Storm drains, sewers and water mains are being laid near the Hall of Science which are the beginnings of a sanitary system to serve 1,000,000 visitors a day. Working plans are being completed for the States building, which will house the exhibits of 40 states. The Lincoln group nearing completion will be opened to the public this summer. A Chinese Lama temple is now being reproduced. It is finished with copper shingles and covered with gold leaf. The $1,250,000 General Motors building has been started and the Chrysler building will soon be under way. Plans are being drawn for the Sears, Roebuck building and the Illinois building.

L. R. Lohr, general manager, states that 189 contracts have been signed with the foremost industrial corporations of the country for exhibit space. Advance payments of $1,390,000 have been received of the $1,816,000, value of the contracts. He looks forward confidently to full completion well before June 1, 1933.

◆

Municipal Bond Trends

Interest Rates Favorable but Sales and New Authorizations Small

Interest rates during May on municipal bonds backed by good credit conditions continued along the improved favorable lines of April sales (*ENR*, May 12, p. 707). Resales by first buyers showed some yield rates well below the coupon rates, such as 3 to 4.15 for 4¼'s of Albany, N. Y. Contrariwise, Paterson, N. J., on nearly $2,000,000 of 1934-72 bonds is to pay 6 per cent and the resale yield price is 5.6. Philadelphia in the financial doldrums for months past, had offers on June 3 for only $107,500 of a proposed $20,000,000 issue at 5 per cent.

For the entire month of May, *The Daily Bond Buyer* summary shows, municipal and state bond sales totalled $84,885,584, or well under the previous low record of $102,380,755 in May, 1923, and only half the sales for May, 1931. For the first five months of 1932 the sales were $410,875,349, compared with $729,489,543 in 1929, and with $445,046,932 in 1928.

New bond issues were voted on in 49 communities during May, with 36 adoptions and 13 defeats. The adoptions totaled $10,169,389, but four issues provided all but $919,389 of this total. The 13 defeats applied to a total of $1,862,000, most of which were in three communities.

Asphalt Substitute Developed

The development of a substitute for asphalt at the Prague Gas Works is reported to the Commerce Department by the trade commissioner located at Prague, Czechoslovakia. The substance is composed principally of coal tar and a special kind of crushed stone. A company has been organized to undertake the manufacture of the new material for experimental use in city streets and on country roads outside the city. Czechoslovakia has no domestic source of asphalt.

◆

New Orleans Drainage Canals to Be Enlarged

Because of recent heavy rainfall, which overtaxed the capacity of the drainage system of New Orleans, steps are being taken by the city sewerage and water board to enlarge the two major drainage canals at an expenditure estimated at $200,000. Work of enlarging the Orleans outfall canal will be started immediately and will cost about $25,000. The major project will be the enlargement of the London Ave. canal, which will cost about $175,000. Work on this project will be started as soon as funds are available.

◆

Construction Contracts

Total value of engineering construction contracts reported for the week of June 9, 1932, compared with figures of a year ago, follow:

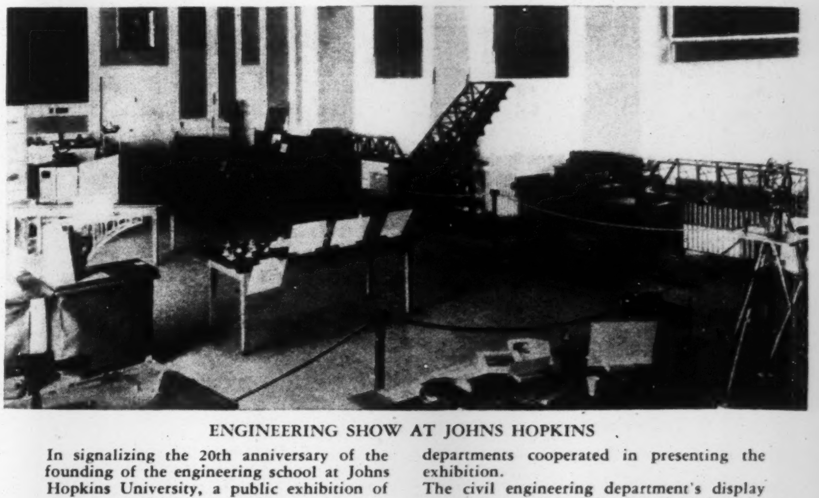
(Thousands of Dollars)

Construction:	Week of June 9, 1932	Average June, 1932	Week June, 1931
Federal government	$21,266	$11,323	$5,039
State and municipal	15,833	12,236	26,394
Total public	$37,099	$23,559	$31,434
Total private	6,788	5,856	17,647
Week's total	$43,887	$29,415	$49,081

Cumulative Construction, Jan. 1 to June 9:

1932, construction	$513,166
1931, construction	1,292,796

Decrease, 60 per cent



ENGINEERING SHOW AT JOHNS HOPKINS

In signalizing the 20th anniversary of the founding of the engineering school at Johns Hopkins University, a public exhibition of the school's laboratory and research equipment was held on May 6 and 7. The object of the exhibition was to acquaint the community at large and especially its younger members with some phases of the various fields of engineering. The civil, electrical, gas and mechanical engineering departments cooperated in presenting the exhibition.

The civil engineering department's display included a series of models depicting the historical development of the railway bridge, an impact machine used in highway research, typical equipment used in water purification and sewage treatment and instruments for the measurement of structural stresses and deflections.

Pavement Repair Prices Fall

Low bids for the repair of asphalt pavement in Buffalo, N. Y., submitted by the Rock Asphalt & Construction Co. for the year beginning July 1, indicate that repair work will be carried out at a cost of $0.435 per square yard. This figure, according to Harry Glassmire, assistant city engineer, compares with 50 cents paid during the present year and the cost of $1.92 per square yard under the plan in effect in the year 1930-31.

Unknown Drain Causes Leak in Water Supply Reservoir

Leakage from LaGrange Lake, the new water supply reservoir for LeRoy, N. Y., which recently caused concern to the municipal authorities, was found upon partially draining the reservoir to be caused by an old line of agricultural drain tile under the dike at the north end of the dam which had not been discovered when the dike was built. The leakage was stopped by plugging the tile with concrete.

Cement Association Warns Against Use of Sugar

The Portland Cement Association has issued a warning against the use of sugar in commonly used portland cement-lime mortars in view of the wide dissemination of a report made to the American Chemical Society at its recent meeting in New Orleans. The statement by the association follows:

> Through an Associated Press release, considerable newspaper publicity has recently been given to the increase in tensile strength of lime mortar briquets by the addition of sugar to the mixing water. The basis for this was a report by Gerald J. Cox and John Metschl, of the sugar fellowship, Mellon Institute of Industrial Research, presented at the April American Chemical Society convention in New Orleans.
>
> The results of tension tests on lime mortar briquets at the age of six months were reported. A 60 per cent increase was obtained in the tensile strength of lime mortar by means of sugar additions up to 6 per cent, by weight, of the lime putty.
>
> A number of different newspapers have rewritten this Associated Press release so that it implies that the addition of sugar to all types of mortars is beneficial. Mr. Cox specifically states: "The description of our work and our recommendations apply only to lime-sand mortar. Our search of the literature has revealed about 70 reports on the effect of the addition of sugar to portland cement. The authors are unanimous in their findings that cane sugar ruins concrete made from portland cement and must not be used."
>
> In view of its recognized harmful effect in portland cement mixtures, the addition of sugar in any quantities to the commonly used portland cement-lime mortars, 1-1-6 and 1-1-4 mixes, should not be attempted.

Personal Notes

Alfred D. Flinn, director of the Engineering Foundation, New York, will receive the honorary degree of doctor of engineering from Worcester Polytechnic Institute on June 17. Mr. Flinn is a graduate of the Institute in the class of 1893.

R. C. Perkins, district engineer of the Arizona State Highway Department, has been appointed deputy state engineer, with office at Phoenix, Ariz., to succeed the late C. C. Small.

Gustave A. Laubenfels, district engineer of the Chicago, Burlington & Quincy R.R., has retired after 55 years service with that road.

F. L. Lindemuth has been appointed chief engineer of the William B. Pollock Co., steel fabricators, Youngstown, Ohio. Mr. Lindemuth formerly was with the Mesta Machine Co., Perkin & Marshall and the Colorado Fuel & Iron Co.

H. V. Schiefer, formerly associated with the R. H. Beaumont Co., is now associated with the C. O. Bartlett & Snow Co. of Cleveland, Ohio, where he will specialize in equipment for bulk material handling.

Fred A. Rogers, professor of physics and electrical engineering at the Lewis Institute of Technology, Chicago, has been appointed dean of the engineering faculty.

W. C. Muldrow, formerly engineer-manager and secretary of Columbia Irrigation District at Kennewick, Washington, is now in the same position with Sunnyside Irrigation District at Benton City, Wash. This is the lower unit of the Sunnyside Division of the U. S. Reclamation Bureau's Yakima Project.

Obituary

Boyd Ehle, civil engineer with Sanderson & Porter, consulting engineers, New York, died at his home at East Creek, N. Y., on June 5, aged 66. Mr. Ehle graduated from Cornell in 1886 and after a few years on railroad construction went to Nicaragua as chief of party for the Nicauragua Canal Construction Co., 1889 to 1893. Subsequently Mr. Ehle was chief of party for the Nicaragua Canal Commission, 1897 to 1899 and then from 18 to 1901 he was principal assistant engine for the Isthmian Canal Commission. is stated that he was the only man wh took part in all the surveys for the tran isthmian canals. From May 1904 to Ma 1905 he was resident engineer on th Chagres division. In addition to his can work Mr. Ehle was active in hydr electric and railway construction both this country and in South America.

J. F. Gilpin, veteran contractor in th Northwest, and for the past three yea president of the Gilpin Construction Co. Portland and Astoria, Ore., died May 2 1932.

George W. Teal, city manager Boulder, Colo., for the past four year died May 30 at his home there after a illness of two years. Mr. Teal was bor in Manchester, England, in 1872 and cam to this country when eight years old. H returned to Europe for his technical edu cation. Later he became prominent i engineering and mining circles in Colorad He was county surveyor of Boulder County for ten years and served as cit engineer of Boulder for eight years.

Seabury G. Pollard, at one time superintendent of the water purification work of Cincinnati and for many years active in the mechanical side of water supply work, died at his home in South Pasadena, Calif., on May 28. Mr. Pollard was a graduate of Cornell University in civil engineering and subsequent to graduation was with the Holly Manufacturing Co. and the Snow Steam Pump Works. Following his work with the city of Cincinnati, he was in consulting practice with J. W. Ellms. About eight years ago he retired from active practice and went to southern California to live.

Fred Brown, for many years active on levee construction on the lower Mississippi River, died suddenly at Lake Charles, La., on May 30. Mr. Brown was a native of Canada and came to New Orleans about 14 years ago as superintendent for the Ewin-Duluth Construction Co.

Julian M. Kennedy, for many years active in engineering for the steel industry, died in Pittsburgh on May 28. Mr. Kennedy recently was the recipient of the Gary memorial medal for conspicuous service to the steel industry.

Alfred B. Nelson, an engineer who before his retirement from active practice, had devoted much of his time to electric railway construction, died in New York on June 4, a few hours after he was taken ill in the office of his son, an engineer located in New York. Mr. Nelson was formerly a resident of Trenton, N. J., but recently has been living on Long Island. He was a graduate of Rutgers College, class of 1879.

John Laing of Hamilton, Ont., formerly one of the engineers employed on the construction of the Welland ship canal, died on May 24, in his 71st year.

John A. Shafer died recently at his home in Newark, Ohio, at the age of 65 years. He was a graduate of the University of Indiana and had been active in engineering and construction work in that state for many years.

SOCIETY CALENDAR

AMERICAN SOCIETY FOR TESTING MATERIALS, annual meeting, Atlantic City, June 20-24.

AMERICAN SOCIETY OF AGRICULTURAL ENGINEERS, annual meeting, Ohio State University, Columbus, Ohio, June 20-23, 1932.

AMERICAN SOCIETY OF CIVIL ENGINEERS, annual convention, Yellowstone National Park, July 6-9.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION, 40th annual meeting, Oregon State College, Corvallis, June 29-July 1.

CONNECTICUT CONFERENCE FOR REGISTRATION OF ENGINEERS AND ARCHITECTS formed in May (*ENR* May 19, 1932, p. 742) held its organization meeting on May 25 and elected Willard S. Conlon president, R. Lincoln Hedlander vice-president, J. S. Kelly secretary, and A. Mausaloff treasurer. The purpose of the organization is to promote public safety through the adoption of registration laws for engineers and architects in the state of Connecticut and also to work for the general welfare of the profession.

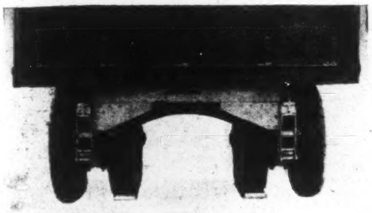
ENGINEERING SOCIETY OF BUFFALO at its annual meeting elected Gideon W. Swan president, Herbert M. Sharp vice-president, Calvin C. Bishop secretary and Bert A. Hansen treasurer. Edward D. Strickland is executive secretary of the organization.

GRAND RAPIDS ENGINEERS' CLUB, at its annual meeting held May 18, elected the following officers: Herbert D. Straight, president, Earl R. Weeber, vice-president, and Bernard Moll, secretary-treasurer.

Construction Equipment and Materials

Twin-Spring Axle for Dual-Wheel Trailer

A new type of twin axle for use with dual-wheel trailers and trucks has been announced by the Fuhrman Trailer Co., Canton, Ohio. The new axle is featured by two full elliptic springs, one on top and one below the axle, with one wheel on each side of the spring. It is claimed that this construction allows both wheels on each individual axle to rotate freely, and if desired, it allows braking on both wheels. It is further claimed that the design allows the tires to conform to road inequalities, giving the same amount of weight on each tire. With an 8-ft.-wide trailer, a distance of 60 in. is provided c. to c. of springs.



Fuhrman axle for trailers.

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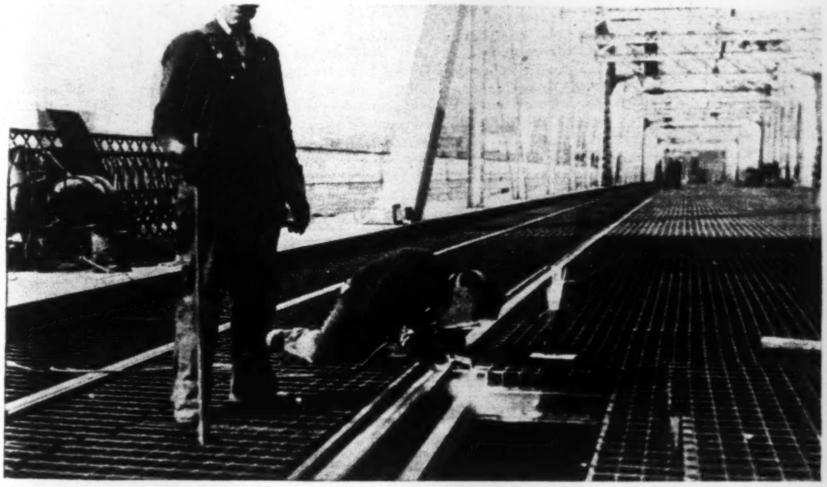
New Bridge-Floor Slab of Steel Tees and Cross-Bars

A new type of bridge-floor construction known as Teegrid is announced by the Truscon Steel Co., Youngstown, Ohio. It is light in weight and has sufficient strength and permanence to support the demands of traffic. The steel Teegrid slab is manufactured from structural tees placed side by side, with their flanges welded together and with triangular cross-bars pressure-welded into the top of the stems of the tees. The standard spacing of the cross-bars is 4 in. The space between the stems of the tees and beneath and around the cross-bars is filled with concrete, a steel-armored wearing surface being presented by the top surface of the cross-bars and the stems of the tees. The steel slabs are manufactured in widths up to 4 ft. and in any desired length. Manufacturing standards provide for an arc weld 2 in. in length located between flanges in the tees and at the ends of the slabs and midway between each support. It is claimed that the welding makes the slab stiff enough for handling and assures lateral distribution of load.

Teegrid floor on bridge in Allentown, Pa.



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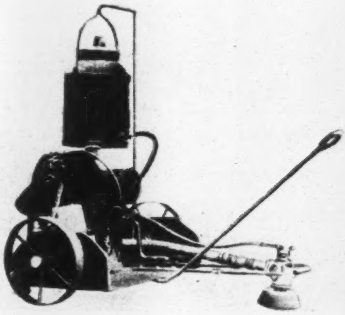
Gas-Driven Concrete Finisher

A 3-hp. concrete bump grinder driven by a gasoline engine is the newest addition to the line of flexible shaft equipment manufactured by the Mall Tool Co., Chicago, Ill. The new tool is used for grinding and finishing cement surfaces and also for such maintenance applications as removing tar and asphalt from rollers and graders. Included in its construction is an adjustable spring-type mounting for the axle spindle, which allows the cup grinding wheel to follow the nature of the surface irregularity. The gas engine operates at 2,600 r.p.m., and speed is increased through the countershaft belt drive to 3,500 r.p.m. at the shaft. Among the attachments available for use with these grinders are a 7-ft. flexible shaft to facilitate grinding on vertical surfaces, and a speed reducer to allow the use of the tool as a gas-engine drill.



Mall Tool Co. concrete bump grinder.

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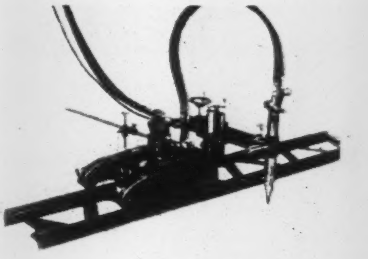
Oxy-Acetyline Cutting Machine

A new oxy-acetylene cutting machine has been announced by the Air Reduction Sales Co., New York City. The new machine is known as the Airco-D-B No. 4 Radiagraph and is a successor to the same company's No. 1-A Radiagraph. The carriage of the machine has been redesigned to give a lower center of gravity and a broader gage. The manufacturer states that there is no tendency for the machine to become unbalanced, even with the cutting torch in its outermost position. Two of the four carriage wheels are swiveled and act as trailers. The machine is driven by a 75-watt variable-speed motor. Either alternating or direct current, 110 or 220 volts, will operate the machine. The speed range of the carriage is from 3 to 38 in. per minute. Circular, as well as straight, cuts can be made.



Broad gage and low center of gravity provided by new carriage design.

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Business Notes

BETHLEHEM STEEL CORP. announces that Lackawanna steel sheetpiling, which it has been manufacturing for years, will in the future be designated as Bethlehem (Lackawanna) steel sheetpiling. Effective May 1 the sale of this product will be handled by Kalman Steel Corp., a subsidiary of the Bethlehem Steel Corp. The Kalman Steel Corp. will also engage in the repurchase and resale of used steel sheetpiling. The Pacific Coast Steel Corp. will handle this product in a like manner on the Pacific Coast.

U. S. BUREAU OF STANDARDS, division of simplified practice, has announced that the revised simplified practice recommendation R-105-32 covering wheelbarrows has received the required degree of acceptance from all interests in the industry and may be considered as in effect as of April 1, 1932. In the original simplified practice recommendation, 41 sizes and types of wheelbarrows were listed; in the revised program this number was further reduced to 27. The division also announces that a proposed modification of simplified practice recommendation No. 57, on wrought-iron and wrought-steel pipe, valves and fittings, by eliminating the 3½-in. nominal inside diameter pipe from table 3, "Double Extra-Strong Pipe," has just been mailed to all interests in the industry for their consideration and written approval. All other diameters and weights in the present draft of the recommendation are to be retained without change.

Current Construction Unit Prices

Two Mid-West Warehouses Built in 1925 and 1927 Repriced to 1932

COST figures are given below of two Mid-Western warehouses erected respectively in 1925 and 1927 and what they would cost today, as estimated by an Illinois contractor. The original estimates have been repriced and the figures are given separately for materials and labor.

"Our wage scale," says the contractor, "was the same in 1925-27 as now." The 16-18 per cent decrease in labor cost is due to increased output and competition for contracts. The decrease is about the same as for materials. Total drop in costs is 18 per cent for the 1925 building and 15 per cent for the 1927 building. The *Engineering News-Record* Cost Index Number averaged 206 in 1925-27 and 159 in the first quarter of 1932, thus recording a general cost drop of 23 per cent.

TABLE I — COSTS OF CONCRETE WAREHOUSE BUILT IN 1925

Six stories and basement reinforced-concrete storage warehouse, round drop panel columns, flat slab, facebrick three sides, stone trim, five-ply tar and gravel roof.

Item	Total	Total	Per Cent Less	Material		Labor	
	8/30/25	4/1/32	1932	1925	1932	1925	1932
General conditions	$26,717	$21,862	18.1	$20,967	$16,112	$5,750	$5,750
Excav. and wreck.	12,880	8,849	31.3	1,157	925	11,723	7,924
Waterproofing	195	140	28.2	78	70	117	70
Fonndations	.....	.....	....	.....	.....	.....	.....
Cement floors	158,181	128,997	18.4	99,873	80,728	58,308	48,269
Reinforced conc	.....	.....	....	.....	.....	.....	.....
Brick masonry	32,401	29,818	8.0	15,003	15,003	17,398	14,815
Cut stone	8,838	6,006	31.9	6,900	4,690	1,938	1,316
Tile fireproofing	215	165	23.2	139	89	76	76
Rough carpentry	106	106	....	48	48	58	58
Millwork	865	865	....	548	548	317	317
Plaster	394	313	20.5	394	313	....	....
Fireproof doors	4,184	3,938	5.8	3,469	3,223	715	715
Steel sash	1,452	1,296	10.7	1,037	1,037	415	259
Sheet metal	386	362	6.2	386	362	.....	.....
Roofing	1,581	1,488	6.4	1,581	1,488	.....	.....
Steel joists and lath	543	493	9.2	495	445	48	48
Scuppers	601	545	9.3	568	512	33	33
Glass and glazing	1,093	953	12.8	1,093	953	.....	.....
Painting, 10% estimated	970	873	10.0	970	873	.....	.....
Structural steel	3,338	2,458	26.4	2,650	1,867	688	591
Miscl. iron	2,968	2,061	30.7	2,555	1,724	413	337
Hardware	93	93	....	93	93	.....	.....
Wiring, 10% estimated	1,725	1,553	10.0	1,725	1,553	.....	.....
Mail chute	12	12	....	12	12	.....	.....
	$259,738	$213,246	...	$161,741	$132,668	$97,997	$80,578
		17.8% less.			18.0% less.		17.7% less

Total cubical contents, 1,660,029 cu.ft. 1925 cost per cubic foot, 15.6c. 1932 cost per cubic foot, 12.8c.

TABLE II — COSTS OF MILL CONSTRUCTED WAREHOUSE BUILT 1927

Four stories and basement. 72x73-ft. Brick walls, concrete columns, beams basement, concrete slab first floor. Lally columns above, 14x14-in. girders, 4x12-in. joints, 2-in. D & M sheeting floors, facebrick, stone belts one side. Stone sills and coping, steel sash and wiring.

Item	Total	Total	Materials		Labor		Total Per Cent Less 1932
	1927	1932	1927	1932	1927	1932	
General conditions	$4,612	$4,612	$3,422	$3,422	$1,190	$1,190	....
Wrecking and excavation	1,847	1,273	1,300	946	547	327	31.0
Foundations	686	512	438	334	248	178	10.0
Cement floors and walks	1,047	955	623	527	451	428	11.0
Reinf. concrete	3,491	2,614	2,113	1,547	1,378	1,067	25.1
Brick masonry	18,676	16,445	9,392	8,796	9,284	7,649	12.0
Rough carpentry	5,915	5,072	4,285	3,622	1,630	1,450	14.2
Millwork	498	432	319	288	179	144	13.3
Lath and plaster	683	529	683	529	.....	.....	22.5
Fireproof doors	4,722	4,059	4,353	3,716	368	343	14.4
Steel sash	433	433	395	395	38	38	....
Sht. mtl. and roofing	1,558	1,215	1,558	1,215	.....	.....	22.0
Glass	403	376	403	376	.....	.....	6.6
Painting, 10% estimated	500	450	500	450	.....	.....	10.0
Structural steel	5,000	3,626	4,463	3,089	537	537	27.5
Miscl. iron	436	358	392	314	44	44	18.0
Hardware	200	200	200	200	.....	.....	....
Wiring, 10% estimated	960	864	960	864	.....	.....	10.0
	$51,667	$44,025	$35,799	$30,630	$15,894	$13,395	....
		14.7% less		14.3% less		15.8% less	

Total cubical contents, 374,100 cu.ft. Cubic foot cost 1927, 14.2c. Cubic foot cost 1932, 12.0c.

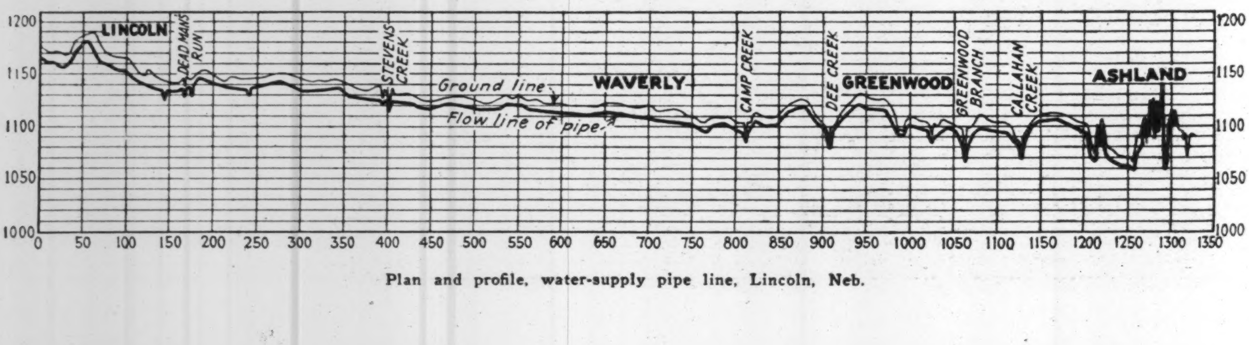
* * *

25-Mile Cast-Iron Water Line for Lincoln, Neb.

LINCOLN, Neb., is laying a 36-in. cast-iron water line 132,600 ft. long, extending from Platte River to the city. Contract was awarded in September, 1931, to Abel & Dobson for $1,309,000. The pipe line is part of a water project to cost nearly $2,000,000. Present supply is from 32 wells in five groups located within the city limits. From 1920 to 1930 population increased from 55,000 to 80,000, and water consumption from 2.14 to 7.8 m.g.d.

Bids were asked also on steel and reinforced-concrete. The lowest three were: steel, Woods Bros. Construction Co., $999,305; concrete, Abel & Dobson, $1,004,000; cast iron, as above. Cast iron was selected because of the preference of the water advisory board for that material. Abel & Dobson submitted the only bid on concrete. Complete list of the other bids is given in the tabulation. H. K. Burket is chairman of the board. D. L. Erickson is city engineer.

Cast-Iron Pipe	Per Foot	Amount
Abel & Dobson, Lincoln, Neb., contract	$9 – $10.30	$1,309,000
U. S. Pipe & Foundry Co., Burlington	8.99 – 10.33	1,315,654
Missouri Engineering & Contracting Co., St. Louis	9.26	1,341,500
Roberts Construction Co	10.00	1,358,700
Peter Kiewet Sons; R. S. Morrow & Son, Omaha	11.37	1,557,471
Williams Bros., Inc., Tulsa	11.42	1,615,050
Newberg & Bookstrom, Lincoln	12.40	1,555,122
J. J. Hannighen Co., Omaha	13.30 – 15.00	1,877,000
Steel Lock Bar ⅜-In. Wrapped Pipe Covering		
Boyle Construction Co.; Massman Co.; Whitmire Construction Co., Kansas City	8.45	1,150,000
Peter Kiewet	8.65	1,195,000
Williams Bros	9.78	1,321,770
J. J. Hannighen	10.75	1,512,000
Steel Lock Bar ¼-In. Without Wrapped Pipe Covering — 2 to 4 per Cent Increase over ⅜-In.		
Steel Electrically Welded ⅜-In. Wrapped		
Wood Bros. Construction Co., Lincoln, low	7.27	999,305
Boyle Construction Co., etc., second low	7.90	1,075,000
Steel Electrically Welded ¼-In. Without Wrapping		
Pittsburgh-Des Moines St. Co., Des Moines	8.00	1,118,129
Boyle Construction Co., etc	8.05	1,100,000



Plan and profile, water-supply pipe line, Lincoln, Neb.